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AUGUST 1956 VOL. LI. NO. 8

FIFTY-FIRST YEAR OF PUBLICATION

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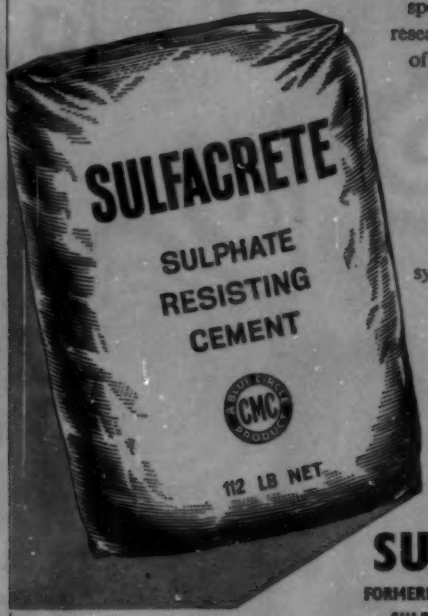

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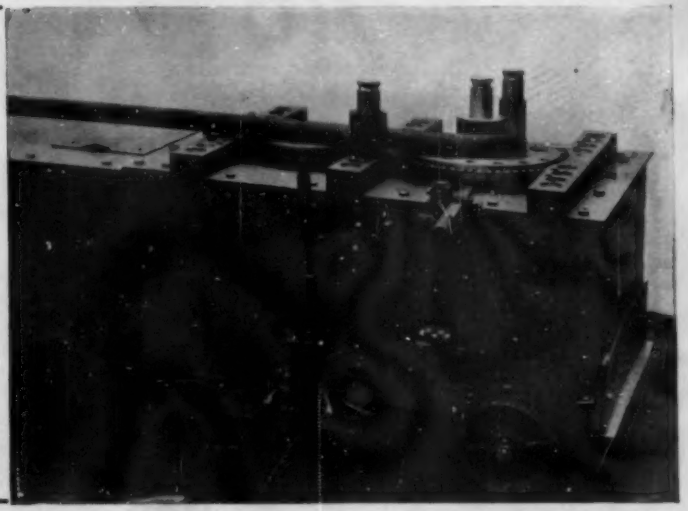

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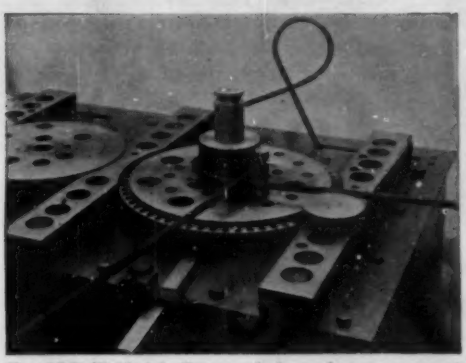

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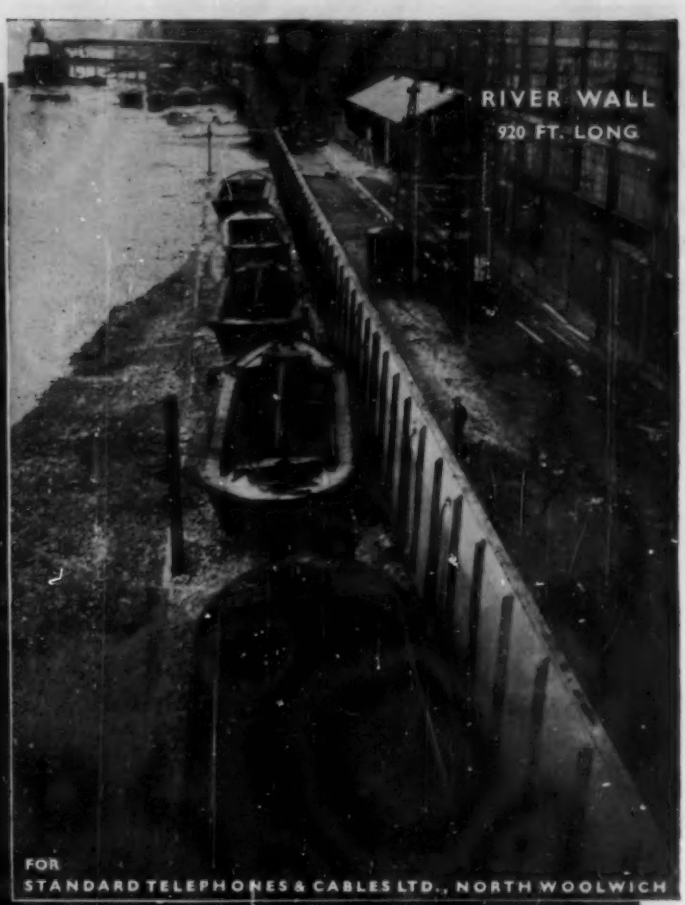

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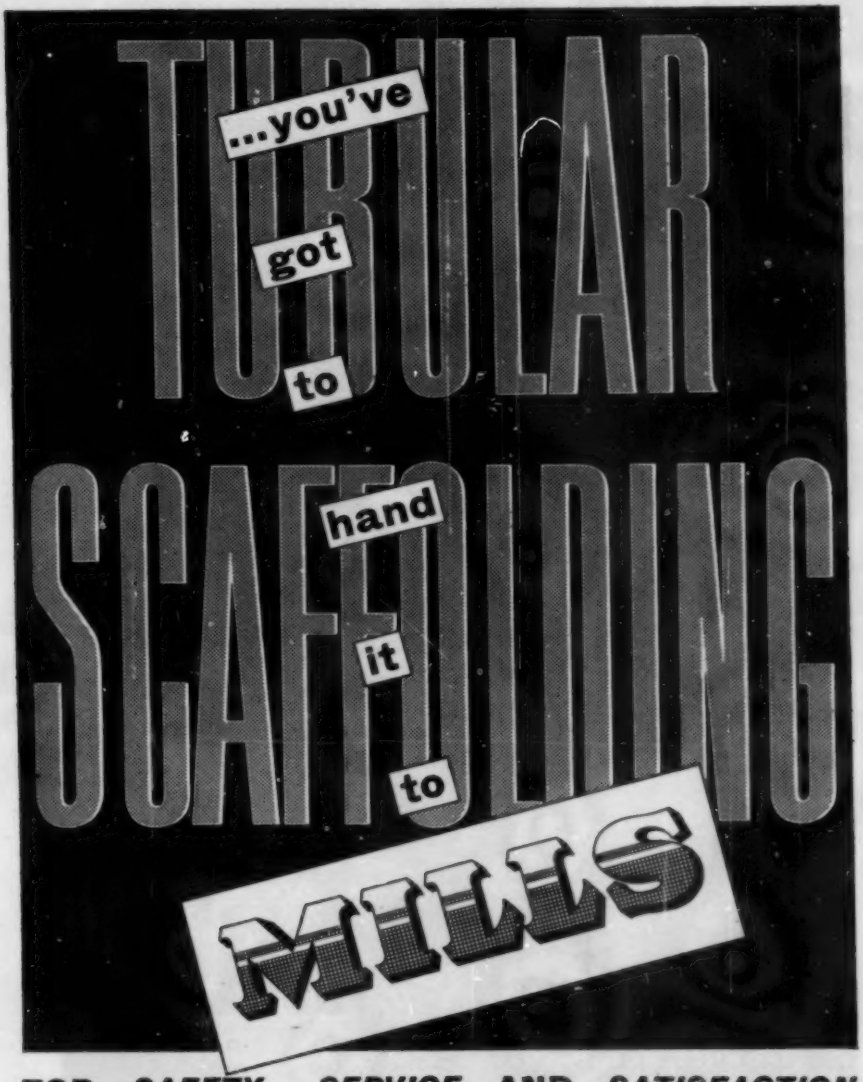

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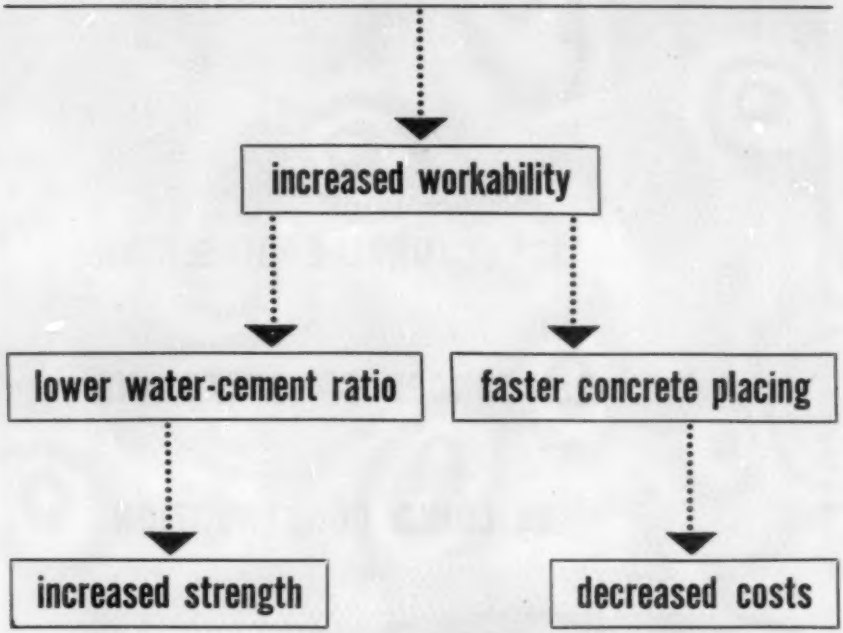

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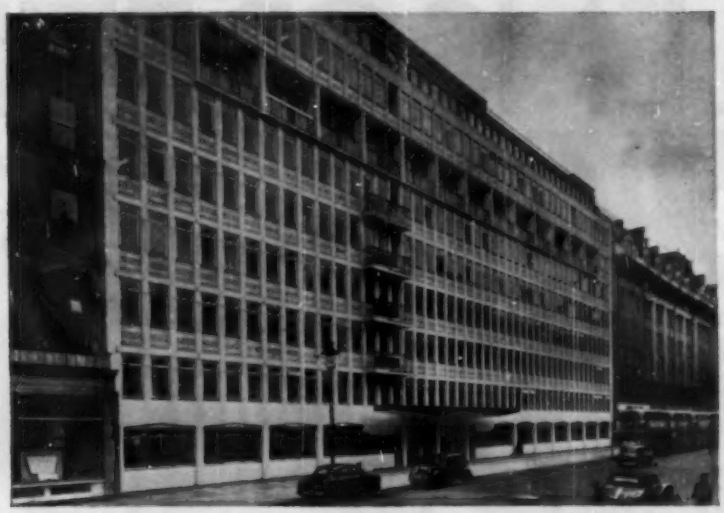

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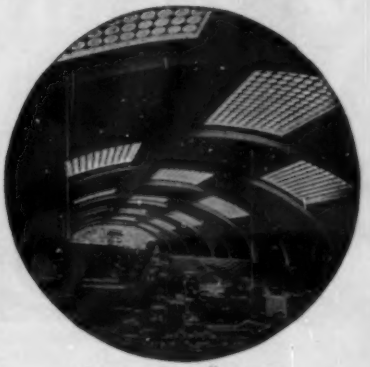

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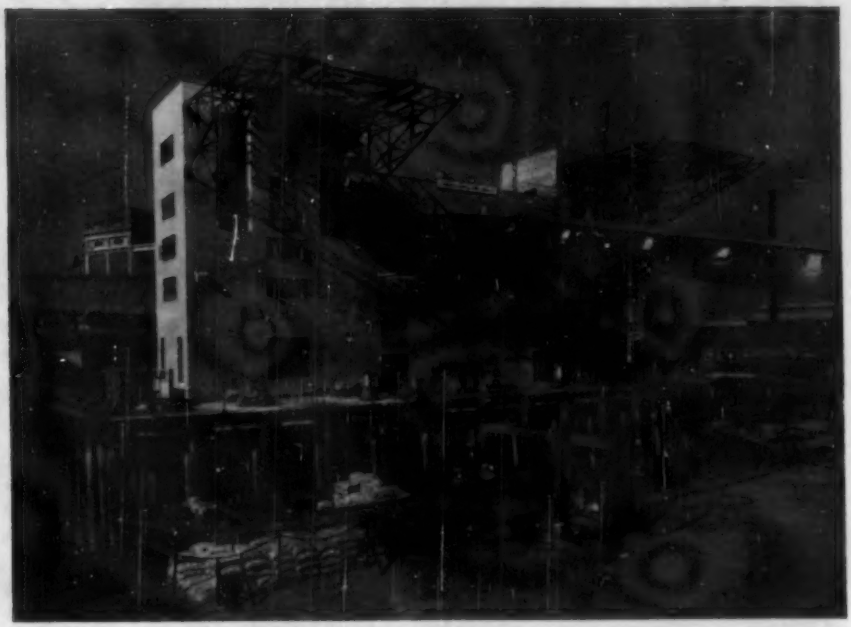

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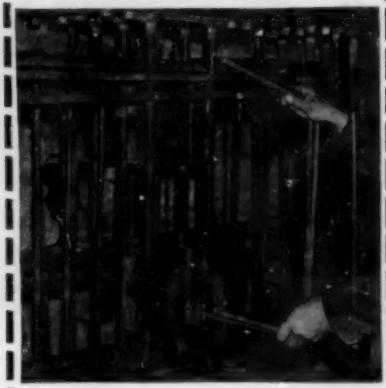

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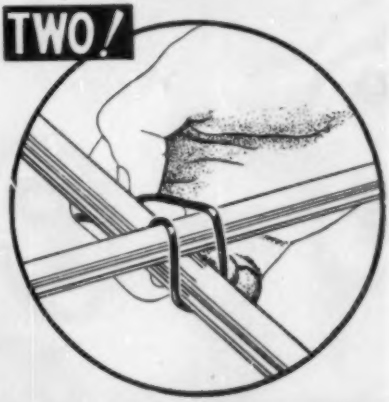

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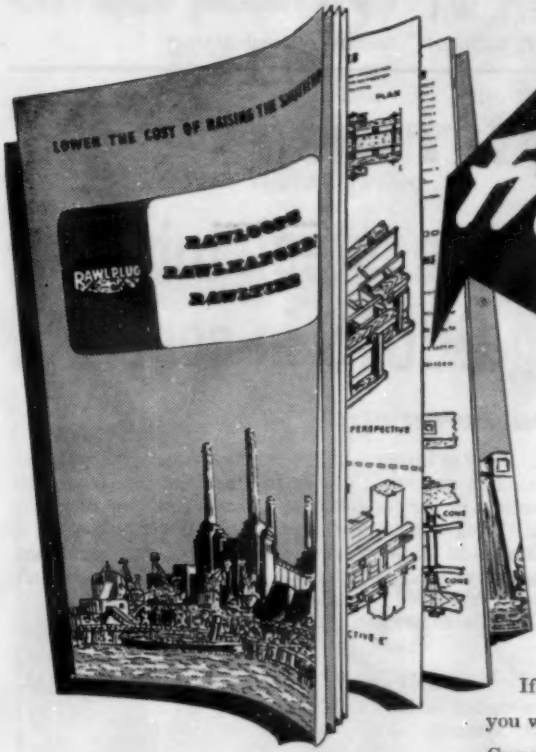

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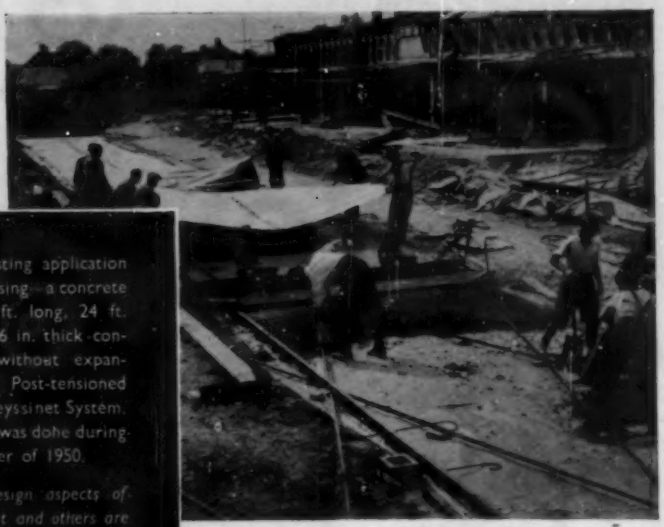

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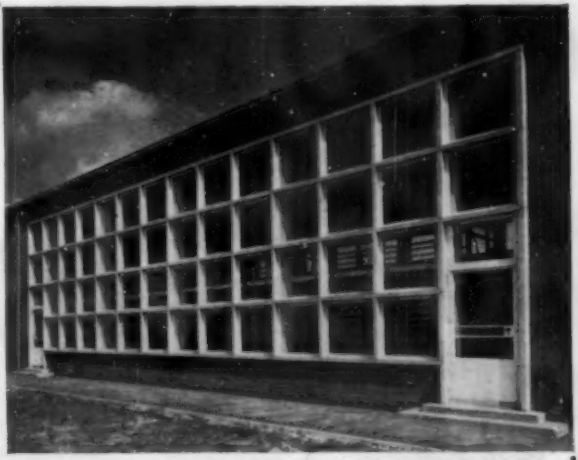

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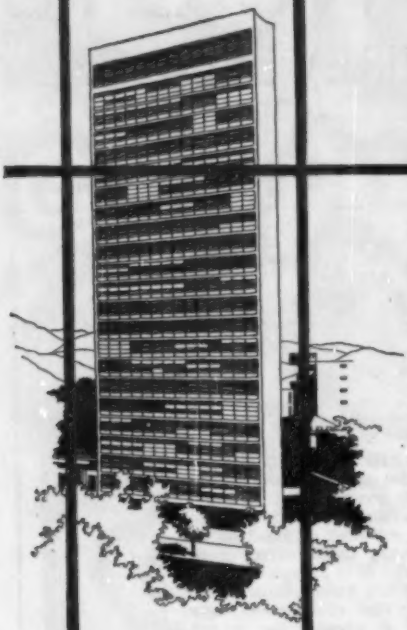

Day by day, MAXWELD reinforces it: reinforces concrete and brickwork, reinforces its own reputation. MAXWELD fabric is made to BSS 1221 Part A—and closely controlled for quality from raw material onwards.
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He'll tell you all about MAXWELD and what it can do for you. He'll tell you what type you need, how much you need, and how much it's going to cost you. He's backed by *Richard Hill's Design Service*, always ready to work out detailed plans and estimates. He's backed by a first class product —and a first class delivery service. Call up the MAXWELD man—at Middlesbrough, London, Birmingham, Manchester, Leeds, Bristol or Glasgow.

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Measurement of sand by volume is always inaccurate unless the bulking due to moisture is taken into account, and this may vary from day to day. A cubic yard may contain anything between 27 and 19 cubic feet of actual sand.

Specified proportions of concretes and mortars measured by volume can never be adhered to unless allowance is made for the variable bulking of sand. A specified 1 : 2 : 4 mix will contain 2 parts of sand if the sand is dry; if the sand is damp it may contain 1½ parts of sand only, and the concrete will be undersanded and less dense and will contain more cement per cubic yard than is called for in the specification.

THE 'RAPID' SAND TESTER enables the concrete maker to know how much sand is contained in a cubic yard. It enables the same amount of sand to be used in every batch of concrete or mortar. It is simple to use, and a test can be made in a few minutes. The calibrated scale shows at a glance the exact amount of damp bulked sand that will give the required amount of actual (dry) sand.

Full details of the apparatus, and examples of the accurate measurement and better and cheaper concrete it makes possible, sent on request.

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"The Rapid Sand Tester has proved its usefulness in the past, and I have pleasure in enclosing an order for twelve more."

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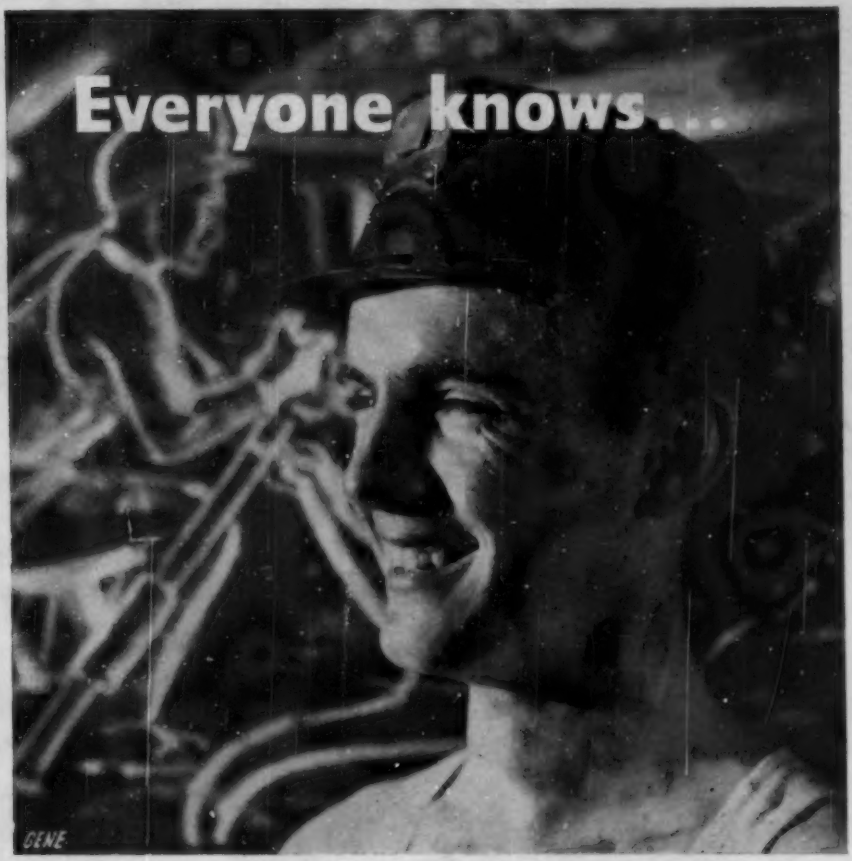

● ● ● that wire is a basic commodity. The miner relies on the wire rope that lowers him to the face, and on the wire from the electrical exploder to the detonator to set off the small controlled explosions that bring coal to industry and our own fireside.

SPENCER wire finds its way even into the bowels of the earth.

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The Johnson "Multibrator" is a high-frequency immersion type concrete vibrator with a brushless driving motor incorporated in the actual 2½" diameter poker (thus eliminating the need for a flexible shaft). The built-in motor runs at a synchronous speed of 12,000 r.p.m. utilising 200 cycles per sec., 3 phase, A.C. Vibrator amplitude is ⅛"—developing a centrifugal force of 1,600 pounds. High frequency and amplitude combine to avoid segregation of mix, and outputs of 25 cu. yds. per hr. can be placed. Other advantages include: low voltage (125 V, 3 phase) for safe operation—entirely waterproof, foolproof switches and "Nettle" type cable connectors—robust motor—2½" dia. poker consolidates, between reinforcing rods—up to 300′ of 3-core tough rubber cable can be attached to each vibrator, and four vibrators can be operated by one power unit sited out of harm's way! Simple to operate—no maintenance—just "cleaning down" required. Supplied with Frequency Changer where current is available or with diesel engine driven Alternator where necessary.

The Multibrator's power unit is conveniently built within a protective tubular frame having wheels for portability. The whole unit has been designed with the accent on robustness and dependability. Photograph by permission of John Mowlem & Co. Ltd., constructional engineers; Trehearne & Norman, Preston & Partners, architects; Arthur Street (London) Investment, Ltd., principals.

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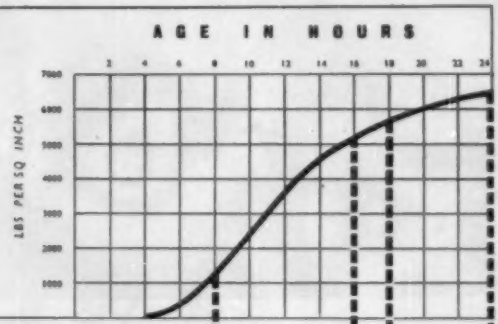

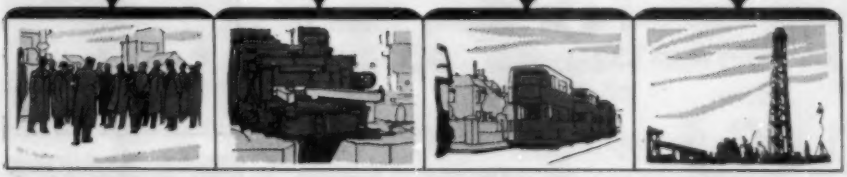

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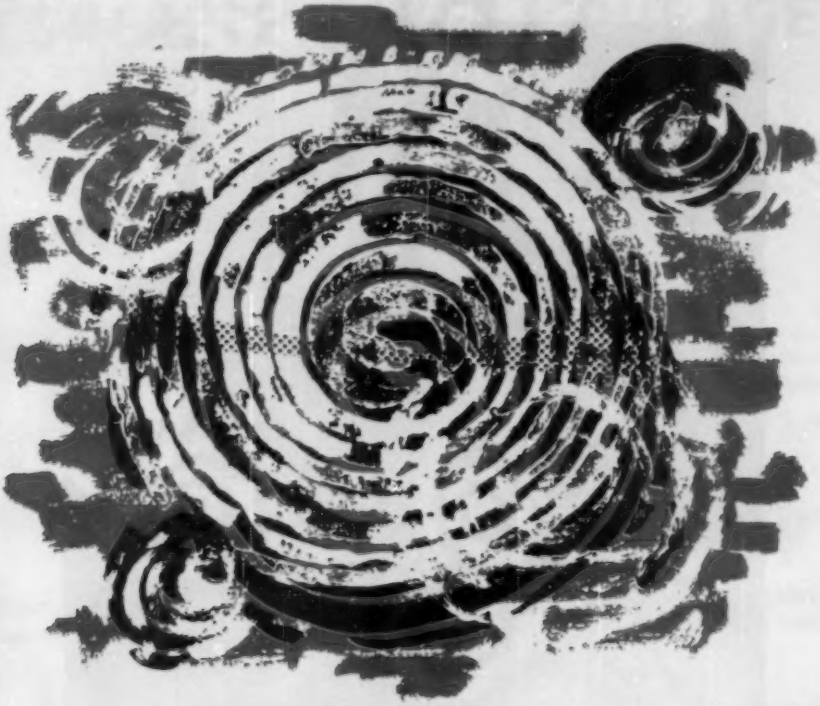

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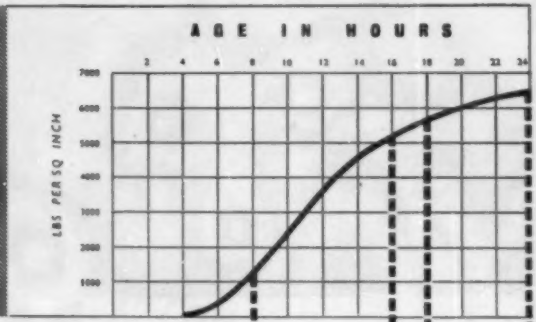

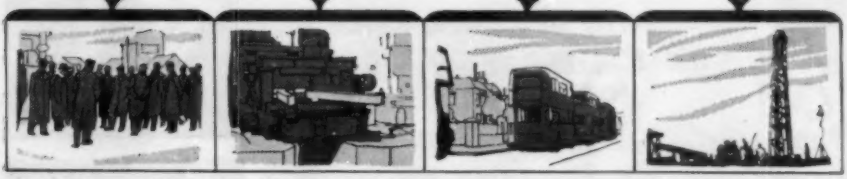

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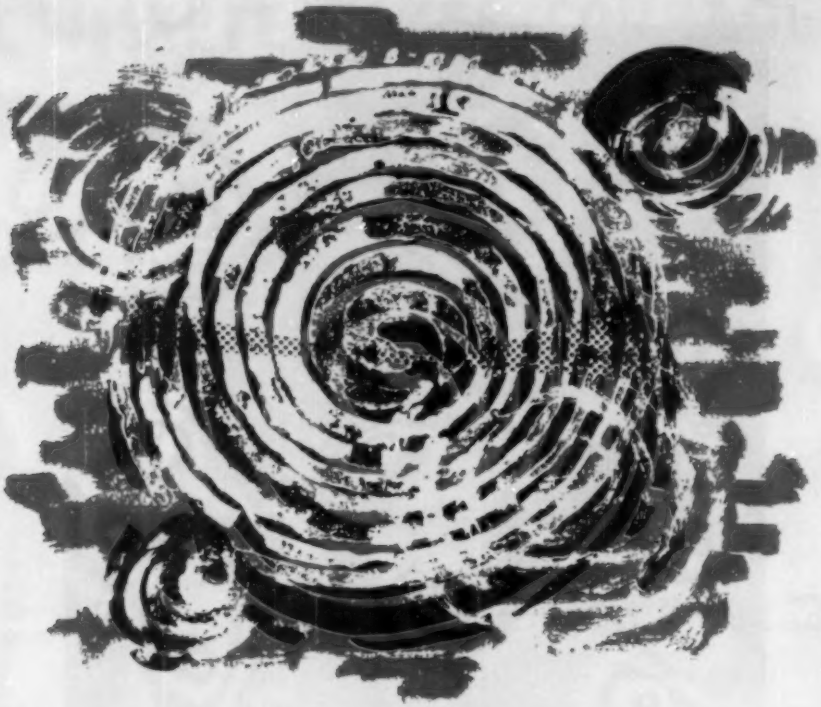

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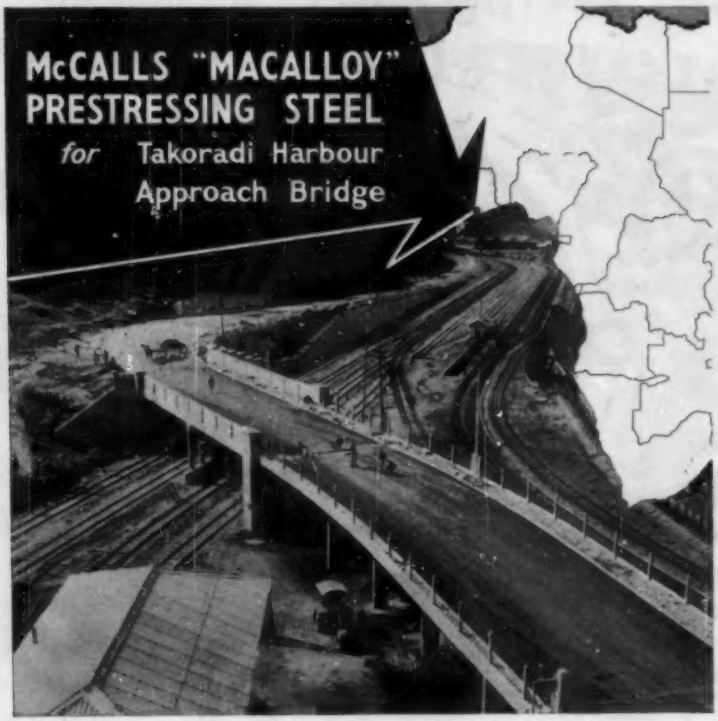

The photograph shows a prestressed concrete road bridge over the railway yard at Takoradi Harbour extensions, Gold Coast Colony. Designed by Rendel, Palmer and Tritton, Consulting Engineers to the Gold Coast Railways and Harbours Administration, and making use of the Lee-McCall system, the bridge was constructed by Taylor Woodrow (West Africa) Ltd. The precast beams of the bridge are of 44 ft. span, each prestressed with 3–1⅛″ diameter Macalloy post-tensioned bars.

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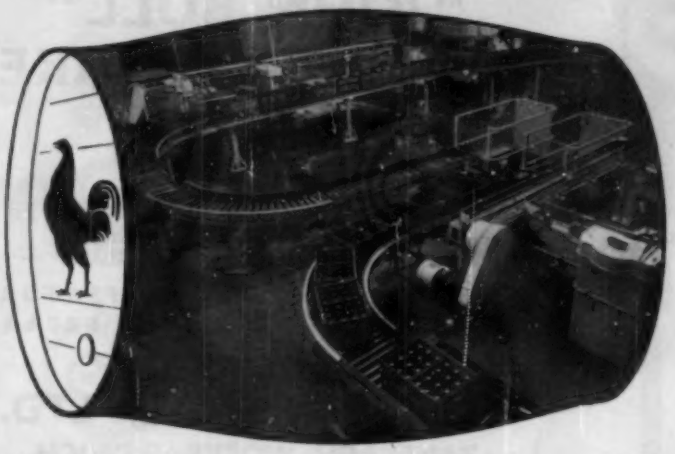

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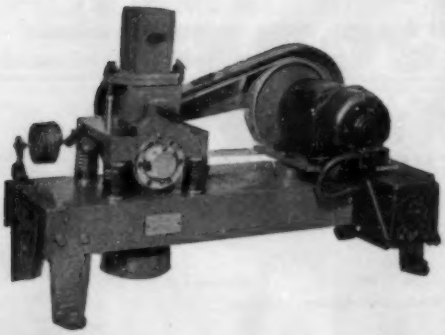

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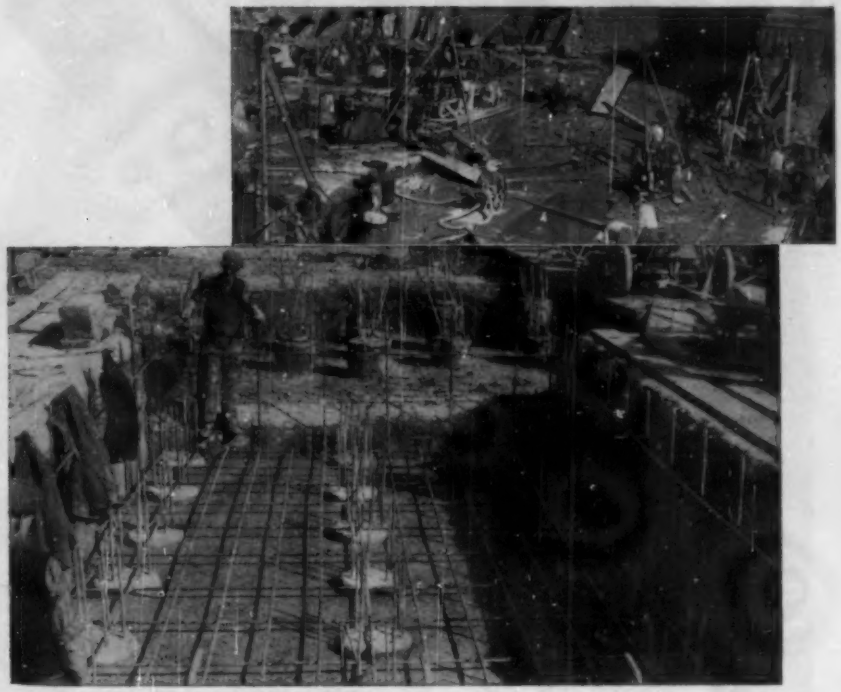

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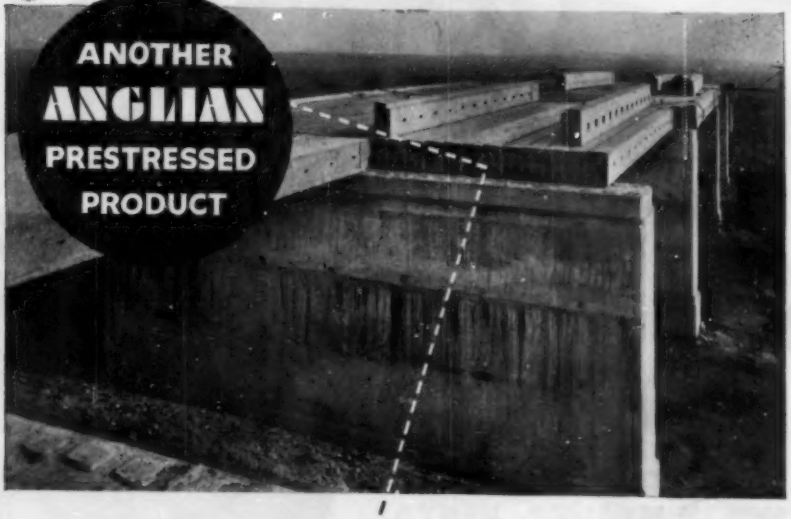

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CONCRETE AND CONSTRUCTIONAL ENGINEERING

INCLUDING PRESTRESSED CONCRETE

Volume LI, No. 8. LONDON, AUGUST, 1956.

EDITORIAL NOTES

Abstract and Relative Terms.

INTELLIGENT discussion is impossible—or at least difficult—if words are used that mean different things to different people, and which are therefore meaningless unless they are defined. Because the clear thinking that is so important in engineering and science is impossible if abstract and relative terms are used without a definition of their meaning, a few words on this subject may not be out of place in connection with the letter printed on page 469 of this number.

Abstractions such as freedom, democracy, justice, status, and so on, are particularly liable to misuse, and there have been several notable examples recently. For example, the Prime Minister described our system of government as a free democracy, without defining how this differed from the converse, which would be such a monstrous contradiction as an enslaved democracy. In this case the word "free" may sound important but has no meaning at all. It was no doubt intended to indicate the difference between a state in which an elector has the choice of voting for candidates of different parties and a state in which only one party is allowed to nominate candidates, but such totalitarian states are not democracies.

The Bill to abolish the hanging of murderers passed its third reading in the House of Commons by 152 votes to 133, that is fewer than a quarter of the Members voted for it. If this Bill becomes law it will be described as an example of a democracy expressing its will; in fact it is nothing of the sort, for it may be that the same proportion of the people, namely one in four, would be in favour of it. In the debate on some sentences of courts of law in Kenya, speakers for both the Government and the Opposition referred to these sentences as the administration of justice. None of them meant what he said. The Government spokesmen were defending the administration of the law and the Opposition were attacking the administration of what they considered to be injustice. The difference that can be placed on the meaning of justice is shown by the well-known argument derived from ancient Greece: If a man steals bread for his children who, through no fault of his, are starving, the law says that he must be punished, but justice says that he should be given more bread to prevent his children from dying of starvation.

These examples show that our correspondent is in good company in demanding status without defining what he means. Status means a person's importance

or social position relative to others, and is meaningless unless it is defined. Our correspondent claimed for engineers, " Give them professional status ". By describing them as professional engineers he agrees that engineers already have what he claims for them. No doubt he means that their status should be raised, and he gives two indications of what he has in mind. One is his complaint that few people would include engineering in a list of the leading professions, but he does not say what he considers to be the leading professions. The other indication is the claim that more engineers should be appointed to the boards of engineering companies. Some engineers are already directors of their companies and no doubt many more could be usefully so employed, for the engineering industries employ more professional men than any other industry. It seems, therefore, that the claim is not that engineers be granted a professional status—this they already have—but that more engineers should be appointed directors of companies and that all should be held in higher esteem by " the man in the street ". This is an abstract way of saying that everyone must be taught to regard engineering as one of the leading professions. We are not told how this is to be done. More directorships will not do it. Extensive and expensive propaganda on the lines of " So and so is good for you " might be effective, but what will have been achieved when the rest of the population has been drilled to recognise that engineering is one of the leading professions ? Suppose that architects, surveyors, and members of other professions make the same claim ; shall we not reach a state where everyone is somebody and no one is anybody ? Are the views of the " man in the street " so important ? This abstract gentleman is often described as the average man, whose views are of little use to anyone except to politicians at election time. If we want sound views we seek the best brains rather than average brains.

Our correspondent suggests that if more young men are to become engineers they must be offered more public esteem and more chance of directorships. We do not believe it. Young men become engineers because they want to be engineers, because to-day engineering offers a financial reward as good as that to be had in most other professions in which the demand exceeds the supply and in which good conditions and good salaries are being offered in order to attract their services. Among manual works status is expressed in " differentials " by which the higher a man's skill the higher his wages, and he uses the extra money to buy more comforts and amenities and so improve his living conditions, and perhaps his status, compared with the less skilled. We have a feeling that most salaried engineers would prefer higher salaries to a public esteem that butters no parsnips, or a very remote chance of being one of the fortunate few who may become directors of trading companies, or even, as has been claimed by the Guild, that its members should be accepted as attestors to applicants for passports. The Guild has now discovered that the signatures of Service pensioners may be witnessed by a variety of people ranging from Members of Parliament to Civil servants with a salary of £200 a year. This is claimed to be a " constant source of irritation to professional engineers ", and the matter has therefore been taken up with the Treasury so that the members of the Guild may enjoy the same status as a junior office girl. It is not easy to see the difference between this claim for " status " and childish snobbery.

Analysis of a Frame with Three Bays.*

By V. A. MORGAN, M.Eng., A.M.Inst.C.E.

A GABLE frame with three or more bays is best analysed by the moment-distribution method. In such a frame with fixed column-bases a general solution would be cumbersome, and a particular solution only is given. The same method can be applied to a larger number of bays and also to hinged bases, using stiffness-ratios and fixing-moments corresponding to the fixity at the ends.

Side Load at the Top of an Outer Column (Fig. 1).

By the principle of superposition, the frame shown in *Fig. 2a* subjected to loads of $\frac{1}{2}P$ acting in opposite directions at B and B_1, and the frame shown in *Fig. 2b* subjected to loads of $\frac{1}{2}P$ acting in the same direction, are together equivalent to a frame subjected to P acting at B as shown in *Fig. 2c*. The cases shown in *Figs. 2a* and *2b* are symmetrical, thus simplifying the moment distributions which, by double geometrical progressions, may be summed to give accurate

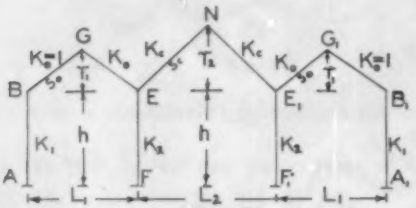


Fig. 1.

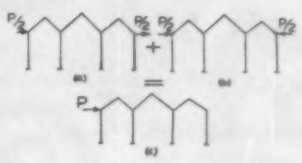


Fig. 2.

instead of approximate results. The summations are given for the two cases. By substituting actual stiffnesses and stiffness-ratios the moments are obtained in terms of the fixing-moments of the columns before distribution. From these moments, thrust equations are obtained from which the final values of all moments and thrusts are obtained. The procedure is similar for frames with hinged bases, suitable column-stiffnesses and moments at the tops of the columns being used.

The notation is as given in this journal for July, 1955. The moment of inertia is I_0, and s_0 is the length of the outer rafters. The stiffness of the outer rafters K_0 is $\frac{I_0}{s_0}$ and is assumed to be unity. Similarly

$$K_c = \frac{I_c}{s_c} \times \frac{s_0}{I_0} = \frac{\text{moment of inertia of central rafter}}{K_0 \text{ times the length of central rafter}}$$

Similarly the stiffness-ratios of the inner and outer columns relative to the outer rafters are $K_1 = \frac{I_1}{K_0 h}$ and $K_2 = \frac{I_2}{K_0 h}$. L_1 and L_2 are the spans of the outer and inner frames measured between the centre-lines of the columns. The height of the column is h, and r_1 and r_2 are the rises of the outer and inner ridges above a line joining B to B_1.

* The analysis of frames of one bay was dealt with by the writer in this journal for July, 1955, and frames with two bays in the number for February, 1956.

Anticlockwise moments are positive. As in the previous article, the fixing moment $D_1 = \frac{3EK_c}{r_1}(\Delta_1 - \Delta_2)$ and $D_2 = \frac{3EK_c}{r_1}(\Delta_4 - \Delta_3)$ for the rafters of the outer bays. The central bay of frame 3*a* has no moment, since the deflections at the tops of columns E and E_1 are both Δ_2, and their relative deflection is zero. For the central bay of frame 3*b*, $D_3 = \frac{3EK_c}{r_2}(\Delta_4 + \Delta_4) = \frac{6EK_c}{r_2}\Delta_4$. Fixing moments at the tops of the columns and at the bases are

$$c_1 = \frac{6\Delta_1 EK_1}{h}; \quad c_2 = \frac{6\Delta_2 EK_2}{h}; \quad c_3 = \frac{6\Delta_3 EK_1}{h}; \quad c_4 = \frac{6\Delta_4 EK_2}{h}.$$

As the frames are symmetrical, distributions for half the frames only are necessary. The central moments carried over are positive in *Fig.* 3*a* and negative in *Fig.* 3*b*, where $a_1 = C_1 - D_1$, $a_2 = C_2 - D_2$, $b_1 = D_1 + C_2$, $b_2 = D_2 + C_4 - D_3$, $x = 7 + 8K_1$, $y = 7 + 8K_2 + 7K_c$. In both cases the moments are the sums of series of the following types.

Case 1.

$\frac{1}{x} \times 1$. $\frac{1}{xy} \times K_c$.

$\frac{1}{x^2y^2} \times (xK_c^2 + y)$.

$\frac{1}{x^2y^3} \times (xK_c^2 + 2yK_c)$.

$\frac{1}{x^2y^4} \times (x^2K_c^4 + 3xyK_c^2 + y)$.

$\frac{1}{x^2y^5} \times (x^2K_c^5 + 4xyK_c^3 + 3y^2K_c)$.

$\frac{1}{x^4y^6} \times (x^3K_c^6 + 5x^2yK_c^4 + 6xy^2K_c^2 + y^3)$.

$\frac{1}{x^4y^7} \times (x^3K_c^7 + 6x^2yK_c^5 + 10xy^2K_c^3 + 4K_cy^3)$.

Case 2.

$\frac{1}{2} \times 1$. $-\frac{1}{xy} \times K_c$.

$\frac{1}{x^2y^2} \times (xK_c^2 + y)$.

$-\frac{1}{x^2y^3} \times (xK_c^2 + 2yK_c)$.

$\frac{1}{x^2y^4}$, etc.

Case 1 can be written

$$\sum \frac{1}{x} + \frac{1}{xy}.K_c + \frac{1}{x^2y^2}.xK_c^2 + \frac{1}{x^2y^3}.xK_c^2 \ldots \text{etc.} \quad . \quad . \quad . \quad . \quad (1)$$

$$+ \sum \frac{y}{x^2y^2} + \frac{2yK_c}{x^2y^3} + \frac{3xyK_c^2}{x^2y^4} \ldots \text{etc.} \quad . \quad . \quad . \quad . \quad (2)$$

$$+ \sum \frac{y}{x^2y^4} + \frac{3y^2K_c}{x^2y^5} + \ldots \text{etc.} \quad . \quad . \quad . \quad . \quad (3)$$

It can be shown that (1) is a geometrical progression whose summation is $\frac{y}{x(y - K_c)}$.

Similarly, the summation of (2) is $\frac{y}{x^2(y - K_c)^2}$. The complete summation is then

$$\Sigma(1) + (2) + (3), \text{ etc.} = \frac{y}{x(y - K_c)} + \frac{y}{x^2(y - K_c)^2}, \text{ etc.} = \frac{y}{xy - K_cx - 1}.$$

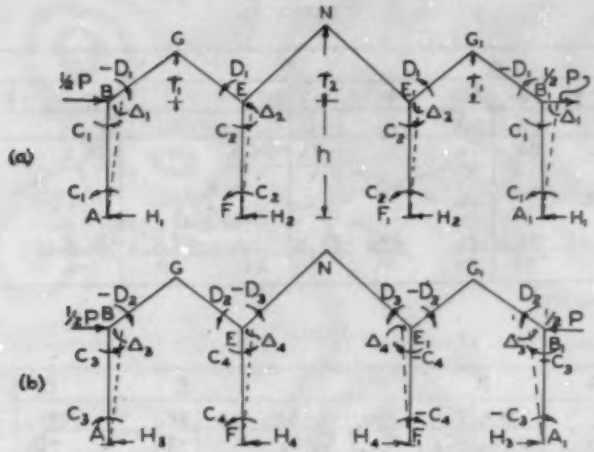


Fig. 3.

For Case 2 the complete summation can be shown to be $\dfrac{y}{xy + K_c x - 1}$.

In this manner the summations of the two moment distributions are as follows.

Case 1.

$$M_{1_{AB}} = c_1 - \frac{4K_1b_1 + 4K_1a_1(y - K_c)}{xy - K_cx - 1}.$$

$$M_{1_{BA}} = c_1 - \frac{8K_1(b_1 + a_1y - a_1K_c)}{xy - K_cx - 1}.$$

$$M_{1_{EB}} = D_1 + \frac{(1 - 7x)b_1 + (y - K_c - 7)a_1}{xy - K_cx - 1}.$$

$$M_{1_{EF}} = c_2 - \frac{8K_2(xb_1 + a_1)}{xy - K_cx - 1}.$$

$$M_{1_{FE}} = c_2 - \frac{4K_2(xb_1 + a_1)}{xy - K_cx - 1}.$$

$$M_{1_{EE_1}} = -\frac{6K_c(xb_1 + a_1)}{xy - K_cx - 1}.$$

$M_{1_{E_1E}}$ do.

$h.H_1 = M_{AB} + M_{BA}$. $h.H_2 = M_{EF} + M_{FE}$.

Case 2.

$$M_{2_{AB}} = c_2 - \frac{4K_1b_2 + 4K_1a_2(y + K_c)}{xy + K_cx - 1}.$$

$$M_{2_{BA}} = c_2 - \frac{8K_1(b_2 + a_2y + a_2K_c)}{xy + K_cx - 1}.$$

$$M_{2_{EB}} = D_2 + \frac{(1 - 7x)b_2 + (y + K_c - 7)a_2}{xy + K_cx - 1}.$$

$$M_{2_{EF}} = c_4 - \frac{8K_2(xb_2 + a_2)}{xy + K_cx - 1}.$$

$$M_{2_{FE}} = c_4 - \frac{4K_2(xb_2 + a_2)}{xy + K_cx - 1}.$$

$$M_{2_{EE_1}} = -D_2 - \frac{8K_c(xb_2 + a_2)}{xy + K_cx - 1}.$$

$$M_{2_{E_1E}} = D_2 + \frac{8K_c(xb_2 + a_2)}{xy + K_cx - 1}.$$

$h.H_2 = M_{AB} + M_{BA}$. $h.H_4 = M_{EF} + M_{FE}$.

The thrusts on the rafters are:

Case 1. In the outer bays, $H_{1_{BE}} = \dfrac{D_1}{2r_1} + \dfrac{3}{4r_1}(M_{1_{BA}} + M_{1_{EB}})$.

In the central bay, $H_{1_{EE_1}} = \dfrac{3}{4r_2}(M_{1_{E_1E}} - M_{1_{EE_1}}) = 0$.

TABLE I(*a*).

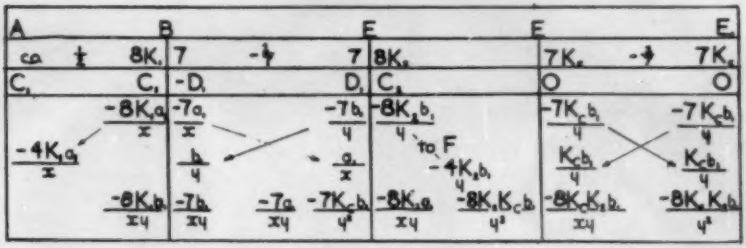
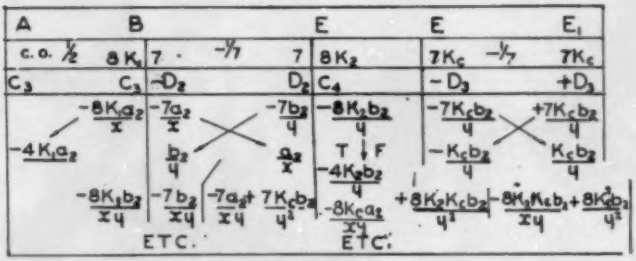


TABLE I(*b*).



Case 2. In the outer bays, $H_{2_{BE}} = \dfrac{D_2}{2r_1} = \dfrac{3}{4r_1}(M_{2_{BA}} + M_{2_{EB}})$.

In the central bay, $H_{2_{EE_1}} = \dfrac{D_2}{3r_2} + \dfrac{3}{4r_2}(M_{2_{E_1E}} + M_{2_{EE_1}})$.

Equations to be solved for c_1 in terms of c_2 for Case 1 are:

$H_2 = H_{1_{BE}}$; substituting c_2 in terms of c_1 in $\frac{1}{2}P = H_1 + H_2$, c_1 and c_2 are obtained in terms of P, and these values are substituted to give the values of the bending moments $M_{1_{AB}}$, etc.

Equations to be solved for c_3 in terms of c_4 for Case 2 are:

$H_{2_{BE}} = H_{2_{EE_1}} + H_4$; substituting c_4 in terms of c_3 in $\frac{1}{2}P = H_3 + H_{2_{BE}}$, c_3 and c_4 are obtained in terms of P, and these values are substituted to give values of the bending moments $M_{2_{AB}}$, $M_{2_{BA}}$, etc.

The final values of the bending moments are:

$$M_{AB} = M_{1_{AB}} + M_{2_{AB}}. \qquad M_{A_1B_1} = M_{1_{AB}} - M_{2_{AB}}.$$

$$M_{BA} = M_{1_{BA}} + M_{2_{BA}}. \qquad M_{B_1A_1} = M_{1_{BA}} - M_{2_{BA}}.$$

$$M_{EB} = M_{1_{EB}} + M_{2_{EB}}, \text{ etc.}$$

As an example, the stiffness of all the members is unity; $r_1 = r_2 = 6$ ft.; $L_1 = L_2$; $h = 14$ ft. Although these are simple values, as the previous equations

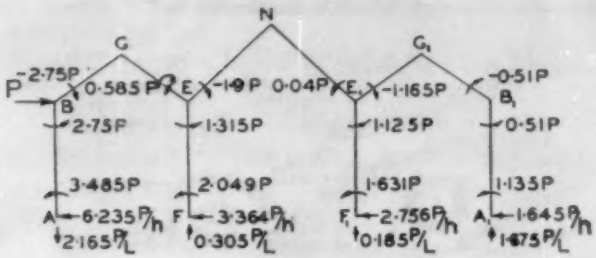


Fig. 4.

are general any variation of stiffness or rise of span can be substituted so long as the frame is symmetrical about the vertical centre-line.

$$D_1 = \frac{3EK_0}{r_1}(\Delta_1 - \Delta_2) = \frac{hK_0}{2r}\left(\frac{c_1}{K_1} - \frac{c_2}{K_2}\right) = \frac{7}{6}(c_1 - c_2).$$

$$D_2 = \frac{7}{6}(c_2 - c_4). \quad D_3 = \frac{7}{6}(c_4 + c_4).$$

$$y = 22. \quad x = 15. \quad xy - K_0x - 1 = 1 = 314. \quad xy + K_0y - 1 = 334.$$

$$a_1 = \frac{7c_2 - c_1}{6}. \quad b_1 = \frac{7c_1 - c_2}{6}. \quad a_2 = \frac{7c_4 - c_3}{6}. \quad b_2 = \frac{7c_3 - 15c_4}{6}.$$

$$M_{1AB} = \frac{970c_1 - 292c_2}{942}. \quad M_{2AB} = \frac{1064c_2 - 292c_4}{1032}. \quad H_{1BE} = \frac{7376c_1 - 6944c_2}{24 \times 942}.$$

$$M_{1BA} = \frac{998c_1 - 584c_2}{942}. \quad M_{2BA} = \frac{1096c_2 - 584c_4}{1032}. \quad H_2 = \frac{1932c_2 - 624c_1}{14 \times 942}.$$

$$M_{1BE} = \frac{584c_2 - 998c_1}{942}. \quad M_{2BE} = \frac{584c_4 - 1096c_2}{1032}. \quad c_2 = \frac{7390c_1}{8914} = 0{\cdot}822c_1.$$

$$M_{1EB} = \frac{728c_1 - 998c_2}{942}. \quad M_{2EB} = \frac{832c_2 - 368c_4}{1032}. \quad H_{2BE} = \frac{8192c_2 - 5264c_4}{24 \times 1032}.$$

$$M_{1EF} = \frac{974c_2 - 416c_1}{942}. \quad M_{2EF} = \frac{1904c_4 - 416c_2}{1032}. \quad H_{2EE_1} = \frac{14032c_4 + 2496c_2}{24 \times 1032}.$$

$$M_{1FE} = \frac{958c_2 - 208c_1}{942}. \quad M_{2FE} = \frac{1468c_4 - 208c_2}{1032}. \quad H_4 = \frac{3372c_4 - 624c_2}{14 \times 1032}.$$

$$M_{1EE_1} = M_{1E_1E} = \frac{24c_2 - 312c_1}{942}. \quad M_{2EE_1} = M_{2E_1E} = -\frac{1536c_4 + 416c_2}{1032}. \quad c_3 = \frac{10971c_4}{2860} \text{ and } c_4 = 0{\cdot}261c_3.$$

$\frac{1}{2}P = H_1 + H_2$. Therefore $c_1 = 2{\cdot}98P$ and $c_2 = 3{\cdot}63P$.

$\frac{1}{2}P = H_2 + H_{2BE}$. Therefore $c_3 = 1{\cdot}225P$ and $c_4 = 4{\cdot}7P$.

Substituting these values, the final solution (see *Fig.* 4) is

$$\left.\begin{matrix} M_{AB} = 2{\cdot}31P + 1{\cdot}175P. \\ M_{A_1B_1} = 2{\cdot}31P - 1{\cdot}175P. \end{matrix}\right\} \quad \left.\begin{matrix} M_{BA} \\ M_{B_1A_1} \end{matrix}\right\} = 1{\cdot}63P \pm 1{\cdot}12P.$$

$$M_{EF} = 1{\cdot}22P + 0{\cdot}095P. \quad M_{FE} = 1{\cdot}84P + 0{\cdot}209P.$$

$$M_{E_1F_1} = 1{\cdot}22P - 0{\cdot}095P. \quad M_{F_1E_1} = 1{\cdot}84P - 0{\cdot}209P.$$

$$M_{EB} = -0{\cdot}29P + 0{\cdot}875P. \quad M_{EE_1} = -0{\cdot}93P - 0{\cdot}97P.$$

$$M_{E_1B_1} = -0{\cdot}29P - 0{\cdot}875P. \quad M_{E_1E} = -0{\cdot}93P + 0{\cdot}97P.$$

To obtain the reactions, calculate moments about E:

$$LV_A = -2{\cdot}75P + 0{\cdot}585P. \quad V_A = -2{\cdot}165\frac{P}{L}.$$

$$LV_{A_1} = 0{\cdot}51P + 1{\cdot}165P. \qquad V_{A_1} = 1{\cdot}675\frac{P}{L}.$$

$$LV_F = -1{\cdot}90P + 0{\cdot}04P + 2{\cdot}165P.$$

Therefore, $V_F = 0{\cdot}305\frac{P}{L}$ and $V_{F_1} = 0{\cdot}185\frac{P}{L}$. The moments at G, N, and G_1 are then obtained by statics.

Side Load at the Top of an Inner Column.

By superposition in the same manner, using the same values of moments in terms of c_1, etc., H_1, H_2, and $H_{1_{BE}}$ are obtained. Solve $H_1 = -H_{1_{BE}}$ for c_1 in terms of c_2 and substitute $\frac{1}{2}P = H_1 + H_2$, remembering that c_2 will be greater than c_1. For the stiffnesses, etc., of the previous example,

$$c_1 = \frac{7390}{9406}, \quad c_2 = 0{\cdot}785c_2, \quad c_2 = 3{\cdot}14P, \quad \text{and} \quad c_1 = 2{\cdot}47P.$$

Similarly, solving $H_3 = -H_{2_{BE}}$ and $\frac{1}{2}P = H_3 + H_4 + H_{2_{EE_1}}$, $c_3 = 0{\cdot}568c_4$ and $c_4 = 0{\cdot}584P$. The final solution is:

$$M_{AB} = 1{\cdot}56P + 0{\cdot}177P. \qquad M_{BA} = 0{\cdot}662P + 0{\cdot}022P.$$
$$M_{A_1B_1} = 1{\cdot}56P - 0{\cdot}177P. \qquad M_{B_1A_1} = 0{\cdot}662P - 0{\cdot}022P.$$
$$M_{FE} = 2{\cdot}63P + 0{\cdot}765P. \qquad M_{EF} = 2{\cdot}15P + 0{\cdot}942P.$$
$$M_{F_1E_1} = 2{\cdot}63P - 0{\cdot}765P. \qquad M_{E_1F_1} = 2{\cdot}15P - 0{\cdot}942P.$$
$$M_{EE} = -1{\cdot}42P + 0{\cdot}059P. \qquad M_{EE_1} = -0{\cdot}73P - 1{\cdot}001P.$$
$$M_{E_1E} = -1{\cdot}42P - 0{\cdot}059P. \qquad M_{E_1E} = -0{\cdot}73P + 1{\cdot}001P.$$

All the forces and moments may then be obtained by statics. These are set out in *Fig.* 5. The distributions of any other set of external forces, such as wind forces or uniform load over the spans, concentrated loads, etc., can then be balanced and solved by means of the distribution of forces in *Figs.* 4 and 5.

A partial check on the moment-distributions in *Figs.* 4 and 5 is provided

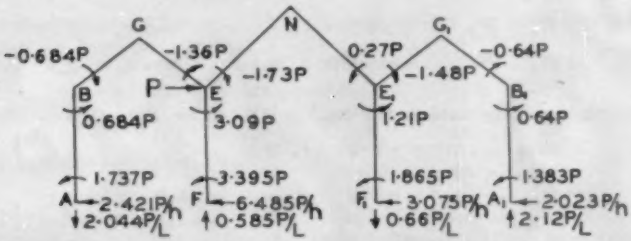


Fig. 5.

by Maxwell's theorem. The deflections at B and B_1 due to a load P at E are proportional to

$$2(1{\cdot}56P \pm 0{\cdot}177P) - (0{\cdot}662P \pm 0{\cdot}022P) = 2{\cdot}46P \pm 0{\cdot}33P,$$

since $$\Delta_B = \frac{(2M_{AB} - M_{BA})h^2}{6EI_{BA}}.$$

Similarly the deflections at E due to P at B and B_1 are proportional to

$$2(1{\cdot}84P \pm 0{\cdot}21P) - (1{\cdot}22P \pm 0{\cdot}095P) = 2{\cdot}46P \pm 0{\cdot}33P.$$

Hinged Column-bases with Side Load.

For frames with hinged column-bases and side loads P at B or E the procedure is similar. The stiffness is modified to $6K_1$ and $6K_2$ for outer and inner columns; $x = 7 + 6K_1$; $y = 7 + 6K_2 + 7K_c$. The terms a_1, a_2, b_1, and b_2 are unaltered.

$$M_{BA} = c_1 - \frac{6K_1(b_1 + a_1y - a_1K_c)}{xy - K_cx - 1} \text{ and } c_2 - \frac{6K_1(b_2 + a_2y + a_2K_c)}{xy + K_cx - 1}.$$

$$M_{EF} = c_2 - \frac{6K_2(xb_1 + a_1)}{xy - K_cx - 1} \text{ and } c_4 - \frac{6K_2(xb_2 + a_2)}{xy + K_cx - 1}.$$

Also $$c_1 = \frac{3\Delta_1EK_1}{h},\ c_2 = \frac{3\Delta_2EK_2}{h},\ c_3 = \frac{3\Delta_3EK_1}{h}, \text{ and } c_4 = \frac{3\Delta_4EK_4}{h}.$$

As M_{AB}, etc., $= 0$, the solution is similar but the work is reduced.

Uniform Load over Three Spans (Fig. 6).

A similar procedure is used for unsymmetrical loads. The fixing-moments are $M = \frac{wL^2}{48}$ unless the central span is different, that is $M_c = \frac{wL_2^2}{48}$. For the case of equal spans and rises the fixing-moments and thrusts balance at E and E_1, and the moments M and $-M$ at B and B_1 only need be distributed. The distribution follows the pattern shown in *Table* II.

Summing gives terms which, if M_c differs from M_1, are almost identical with the second terms in Case 2. However, $a_1 = M$, $b_2 = M_c - M = 0$, and $k_c = 1$, so that

$$M_{AB} = \frac{-4K_1(y+1)M}{xy + x - 1} = -\frac{23M}{86}.$$

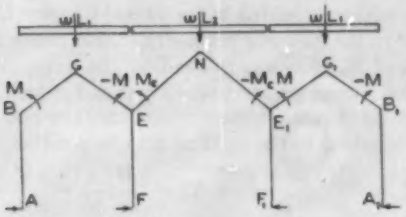
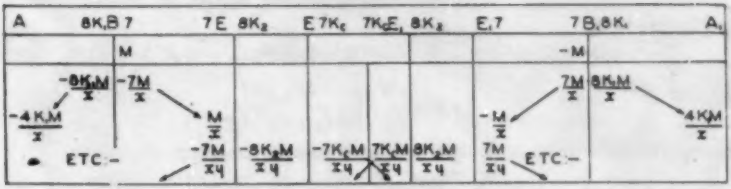


Fig. 6.

TABLE II.



$$M_{BA} = -M_{BE} = \frac{-8K_1(y+1)M}{xy+x-1} = -\frac{46M}{86}.$$

$$M_{EB} = \frac{(y-6)M}{xy+x-1} = \frac{4M}{86}.$$

$$M_{EF} = \frac{-8K_2M}{xy+x-1} = -\frac{2M}{86}. \quad M_{FE} = \frac{-4K_2M}{xy+x-1} = \frac{-M}{86}.$$

$$M_{EE_1} = \frac{-8M}{xy+x-1} = \frac{-2M}{86}, \text{ etc.}$$

The arithmetical values are obtained by cancelling out after substituting K_1 and $K_2 = 1$, $x = 15$ and $y = 22$. $M_{E,E}$, $M_{F,E}$, M_{E,B_1}, etc., have opposite signs to M_{EE_1}, M_{FE}, etc.

THRUST AT B.—This seems best arrived at by assuming that there is a hinge at B resisting the outward thrust exerted by the rafters and the inward force exerted by column BA. The force exerted by the hinge is H_{BE} (inwards) + H_{BA} (outwards).

$$H_{BE} = H_0 + \frac{3}{4r}(M_{EB} - M_{BE}),$$

$$H_0 = \frac{5wL^2}{32r} = \frac{15M}{2r} \text{ inwards } + \frac{3}{4r}\left(\frac{4-46}{86}\right)M = \frac{102{\cdot}5M}{86},$$

and $$H_{BA} = \frac{M_{BA} + M_{AB}}{h} = \frac{-69M}{86h},$$

so that the force exerted by the hinge (since $r = 6$ ft. and $h = 14$ ft.) is $\frac{97{\cdot}32M}{86}$ and equals U_B acting to the right of B.

In a similar manner if it is assumed that there is a hinge at E resisting the thrusts of rafter frames BGE and E_1NE and column EF, then U_E is the resultant resisting force of this hinge. Thus at E the frame BGE exerts an outward thrust from G of H_0, which is resisted by an inward force on the hinge and a thrust of opposite sign due to the bending moments. Also frame ENE_1 exerts an outward thrust from N of H_0, which is resisted by the hinge, and a moment thrust of the same sign. EF causes a force to the right of E which is resisted by a force to the left of E by the hinge.

Assuming a force acting to the right of E to be positive, then

$$U_E = -H_0 + \frac{3M}{4r}\frac{(46-4)}{86} + H_0 + \frac{3M}{4r}\cdot\frac{4}{86} - \frac{3M}{86h} = \frac{5{\cdot}5357M}{86}$$

acting to the right at E.

These forces U_B and U_E and the equal and opposite forces U_{B_1} and U_{E_1} prevent deflection at joints B, E, E_1, and B_1, and the effect of their removal is equivalent to applying equal and opposite forces at these points, that is by adding algebraically the components of these forces (given in *Figs.* 4 and 5) to the forces and moments of the undeflected frame. By algebraically is meant that for force components of U_{B_1} and U_{E_1} the signs and directions are as given in *Figs.* 4 and 5, whilst for U_B and U_E the signs of the moments and directions of the thrusts are opposite to those given in *Figs.* 4 and 5. Thus

$$M_{BA} = \frac{-46M}{86} - 2{\cdot}75U_B - 0{\cdot}684U_E + 0{\cdot}51U_{B_1} + 0{\cdot}64U_{E_1} = (\text{since } U_B = U_{B_1}, \text{ etc.}) - 3{\cdot}07M.$$

$$M_{AB} = \frac{-23M}{86} - U_B(3{\cdot}485 - 1{\cdot}135) - U_E(1{\cdot}737 - 1{\cdot}383) = -2{\cdot}94M.$$

$$M_{EF} = \frac{-2M}{86} - U_B(1{\cdot}315 - 1{\cdot}125) - U_E(3{\cdot}09 - 1{\cdot}21) = -0{\cdot}36M.$$

$$M_{FE} = \frac{-M}{86} - U_B(2{\cdot}049 - 1{\cdot}631) - U_E(3{\cdot}395 - 1{\cdot}865) = -0{\cdot}65M.$$

$$M_{EB} = -M + \frac{4M}{86} - U_B(0{\cdot}585 + 1{\cdot}165) - U_E(1{\cdot}48 - 1{\cdot}36) = -2{\cdot}94M.$$

$$M_{BE_1} = +M - \frac{2M}{86} + U_B(1{\cdot}9 + 0{\cdot}04) + U_E(1{\cdot}73 + 0{\cdot}27) = 3{\cdot}30M.$$

H_{AB} = thrust at $A = \dfrac{6{\cdot}01M}{h}$ (inwards). $\quad H_{FE} = \dfrac{1{\cdot}01M}{h}$ (in direction F_{F_1}).

$$V_A = \frac{24{\cdot}13M}{L}. \quad V_B = \frac{47{\cdot}87M}{L}.$$

Moments $M_{B_1A_1}$, $M_{A_1B_1}$, etc., are opposite in sign to M_{BA_1}, etc.

Substituting $M = \dfrac{wL^2}{48}$, all the remaining moments and forces may be determined by statics.

This method may be extended to any number of frames, and a similar method, using appropriate stiffness ratios and fixing moments, may be applied to frames with hinged column-bases.

Nuclear Power Stations.

Messrs. Balfour, Beatty & Co., Ltd., and Messrs. John Laing & Son, Ltd., have been appointed civil engineering contractors to the A.E.I.–John Thompson Nuclear Energy Co., Ltd., which has been formed by the British Thomson-Houston Co., Ltd., Metropolitan-Vickers Electric Co., Ltd., and Messrs. John Thompson, Ltd., to design, develop, and sell complete nuclear power stations and nuclear reactors. At the A.E.I. research establishment at Aldermaston Court this concern is building Britain's first privately-owned research reactor, and Britain's first oversea order for a nuclear reactor has been received from the West German government. Messrs. W. S. Atkins & Partners are acting as consultants for the civil engineering works and services in connection with these works.

Concrete Churches near Coventry.

OUTSIDE Coventry the housing estates at Bell Green, Willenhall, and Tile Hill are rapidly nearing completion. The need for new churches for these communities was keenly felt by the late Bishop, Dr. Gorton, but funds to cover the conventional type of church, costing about £30,000 each, did not exist. At the Bishop's request, therefore, Professor Basil Spence, O.B.E., A.R.A., prepared designs for churches with a maximum amount of standardization that could be erected at a lower cost. By close co-operation with the contractors, Messrs. George Wimpey & Co., Ltd., it has been possible to produce a church, church hall, and campanile at a cost of about £16,000. Although these buildings are standard, they will be grouped differently on each site, and individuality will be achieved by varying the fenestration and by dressing the belfries with louvres of different patterns and colours. The church at Tile Hill is shown in *Fig.* 1, but the church at Bell Green will have glazed gable walls at both ends, and the church at Willenhall will have a glazed west gable and sidelights to the altar. The churches have been designed to make use of no-fines concrete, and it is hoped that they will be completed early next year.

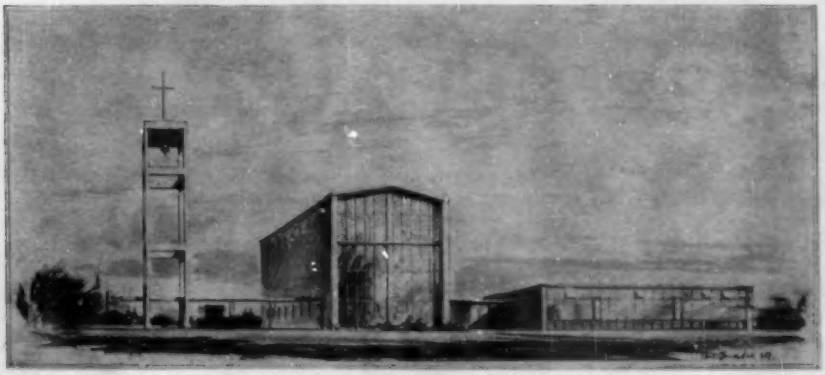


Fig. 1.

The roof of the nave (*Fig.* 2, page 461) is supported by precast reinforced three-hinge frames at 10 ft. centres resting on block foundations 4 ft. 6 in. square by 2 ft. 6 in. thick. Ground-beams span between these blocks to support the no-fines concrete walls. The walls are cast in panels about 9 ft. long, 1 ft. 8 in. thick and 30 ft. high in one pour. It is general practice to render no-fines walls on both surfaces. Here, however, the outside will be finished with spardash and the inside face left exposed. Plywood shuttering will be used instead of the more usual mesh. The walls of the nave are to be decorated with sprayed cement paint. The rough texture is considered appropriate for the large area of the walls, and also provides good sound absorption. Openings for the windows will be formed in the shuttering, and lined with precast concrete frames. The floor of the nave will be on hardcore, and will not be connected to the ground beams.

The tower is a reinforced concrete frame cast in situ. The vertical members are 1 ft. square in cross section, and the horizontal members are 1 ft. 3 in. deep by 1 ft. wide. Pockets are arranged so that precast vertical concrete louvres can be fixed in different positions for each church. These louvres are 2½ in. thick by 9 in. deep, and have a maximum height of 16 ft. Oak plugs, 1 in. square, are cast into the louvres to which stove-enamelled panels can be screwed; these panels will be black, grey, and yellow at Bell Green,

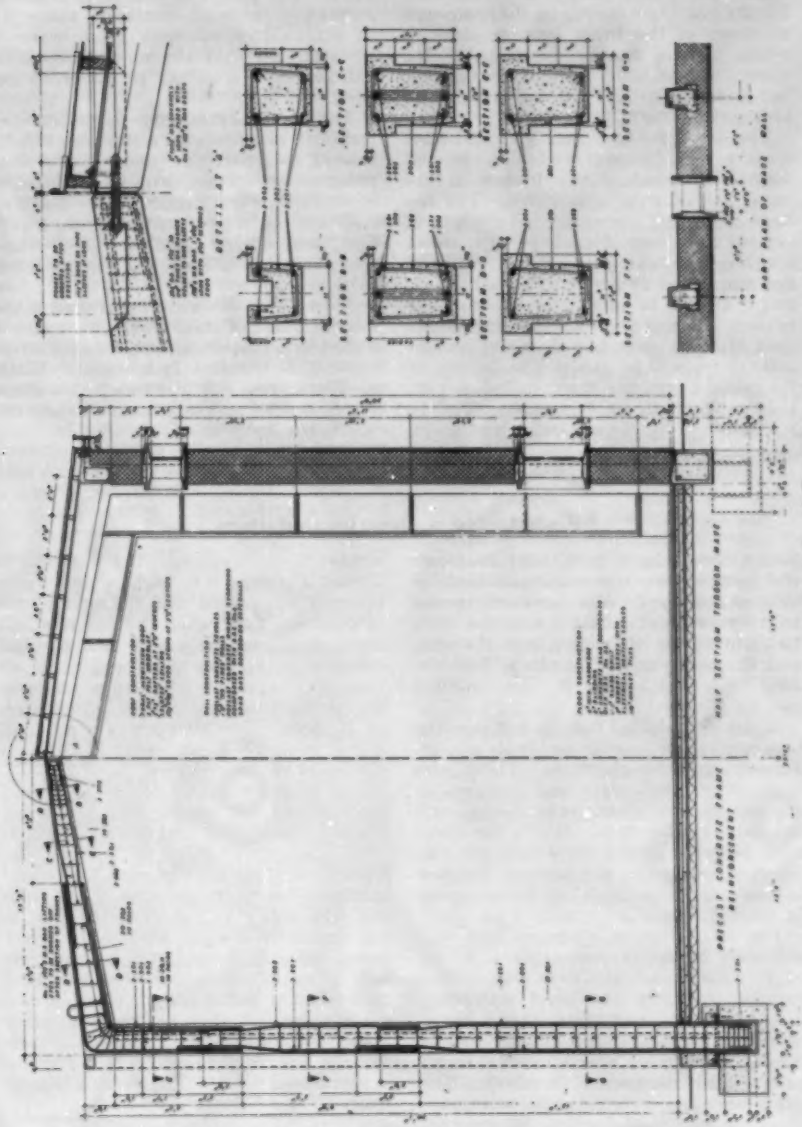


Fig. 2.—Details of Frames.

black, grey, and green at Tile Hill, and black, grey, and purple at Willenhall. Planks 4 in. thick resting on the horizontal members of the tower provide slabs to which ladders can be fixed inside the tower. These slabs are not continuous, but cover only half the tower and are arranged in varying positions.

The column-bases and ground-beams are of 1 : 3 : 6 concrete, and all the precast members, including the frames in the nave, are of 1 : 2 : 4 concrete. The no-fines concrete comprises one part of cement to 8 parts of graded aggregate of which 95 per cent. passes a ¾-in. ring and not more than 10 per cent. passes a ¼-in. ring. It will be mixed for at least one minute in a mixer-drum revolving seventeen times a minute. A water-cement ratio of 0·4 will be used. The density of the no-fines concrete in the walls is 105 lb. per cubic foot, and the compressive strength 350 lb. per square inch at seven days and 460 lb. per square inch at 28 days. The walls 1 ft. 8 in. thick rendered externally have an insulation value (U) of 0·2. Compared with brick walls it is estimated that the no-fines concrete will result in a saving of about 15 per cent.

The external rendering to the no-fines walls will be a first coat comprising 2 parts cement to 1 part lime and 9 parts sand measured by volume, with the addition of a water-repelling agent. The finishing coat will comprise 2 parts cement, 1 part lime, and 9 parts sand, to which white spar chippings will be applied before it has hardened.

A very light roof covering is used in the form of a skin of aluminium alloy which is locked into castellated angle fixing strips when it is unrolled in horizontal bands across the roof. The fixing strips occur at 2 ft. centres and each band overlaps the one below by 3 in.

Resurfacing a Deteriorated Dam.

STEEL sheets have been used to repair the faces of two concrete dams built in California in 1913. The dams are exposed to very severe climatic conditions with frequent cycles of freezing and thawing, and the concrete had in places deteriorated to a depth of 2 ft. The method was as follows.

Holes were drilled 6 ft. to 8 ft. into the concrete at 6 ft. centres and bolts of 1 in. diameter grouted into them. These bolts were fitted with nuts and washers to support the steel sheets at least 6 in. from the face of the dam. While the bolts were being fixed the loose concrete was knocked away, and the greatest distance between the steel and the concrete was in some places 2 ft. 6 in.

The steel was of 10-gauge and was delivered in sheets measuring 6 ft. by 20 ft. Each pair of sheets was welded together to form one sheet measuring 6 ft. by 40 ft., and these were lowered from the top of the dam and secured to the projecting bolts, which passed through holes already formed in the sheet. The joints were lap-welded and the nuts were welded to the washers. The steel sheets formed a shutter 6 ft. high behind which concrete was placed, and the next course of steel was fixed when the concrete had been placed in the course below. The concrete was made with aggregate of ¾-in. maximum size and had a compressive strength of 2000 lb. per square inch at 28 days. The slump was 3 in., and 4 per cent. to 5 per cent. of air was entrained in the mixture.

The exposed surface of the steel was sand-blasted and coated with a coal-tar enamel applied cold, and this was covered with aluminium paint to reduce the absorption of heat. The steel has bulged in places due to the pressure of the fresh concrete, but it is not expected that it will remain in contact with the concrete except when it is subjected to the pressure of water. It is, however, expected that this new facing, together with a new drainage system that has been installed under the dam, will prevent future damage due to freezing. The foregoing is abstracted from "Engineering News-Record" for February 2, 1956.

V-shaped Frames for Supporting Bridges.

By E. BÖLCSKEY.

[This article is a translation of a paper published in "Acta Technica", the journal of the Academiae Scientiarum Hungaricae, of Budapest, by permission of the Academy.]

In *Fig.* 1 are shown examples of bridges with inclined columns compared with bridges supported by vertical columns; at (*a*) is shown to the left a bridge with a beam extending to form cantilevers over two columns, and to the right is the proposed construction with inclined columns; at (*b*) a three-pinned arch is compared with a statically-determinate application of the proposed method; and at (*c*) are alternative methods of construction of a three-bay frame. These examples show some of the applications of the method in which vertical supports subjected to bending and compression are replaced by inclined supports subjected to direct tension or compression only. The analysis is limited to a single-span bridge with cantilevers such as that shown in *Fig.* 1 (*a*). This form of structure is often used for bridges such as those carrying railways over ravines or roads over railway cuttings.

Theory of Two-hinged Frames on V-shaped Supports.

In principle the calculations are similar to those for the normal type of frame with columns subjected to bending. The only difference is that the columns are replaced by inclined members which resist the horizontal and vertical forces by direct compressions and tensions rather than by their resistance to bending; both types of structure are statically-indeterminate in the first degree.

To simplify the calculation, assume that the beam is of constant moment of inertia

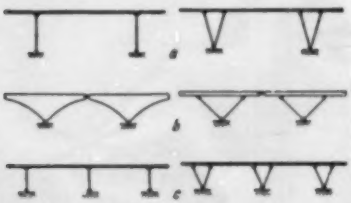


Fig. 1.

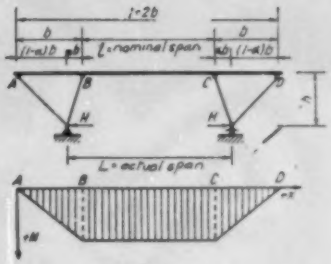


Fig. 2.

and that the influence of the axial deformations of the supports is negligible. *Fig.* 2 shows the primary system of the proposed construction, one of the bases being hinged and the other on rollers so that the statically-indeterminate force may be defined as the horizontal reaction H.

In order to draw the influence line for H, assume that unit horizontal forces act at the level of the hinges at the base. The bending moment on the beam due to these forces is shown in *Fig.* 2. The influence line for H is determined by

$$\eta_{Ah} = -\frac{a_{01}}{a_{11}} = -\left(\frac{a_{01}}{a_{11}}\right),$$

where a_{01} is the deflection due to bending of the beam and a_{11} is the horizontal deflection of the movable support due to the action of a unit force H. The values in parentheses are the product of the deformations and the flexural rigidity EI.

Calculation of a_{01}.

The elastic line, or the deflection-curve, is easily determined from the second integral of bending moment due to the unit horizontal force

$$\frac{d^2y}{dx^2} = -\frac{M}{EI},$$

where y is the deflection, x the position of the section considered with respect to the origin, and M the bending moment. The expression is integrated in two portions, one for the part b of the beam between the inclined supports and one for the span l between the inner legs of the inclined supports.

In the length b:

$$M_b = + \frac{h}{b}x.$$

$$(\phi_b) = - \int M_b \, dx = - \frac{h}{2b}x^2 + c_1.$$

$$(y_b) = \int\int M_b \, dx \, dx = - \frac{h}{6b}x^3 + c_1 x + c_3.$$

In the span l:

$$M_l = + h.$$

$$(\phi_l) = - \int M_l \, dx = - hx + c_2.$$

$$(y_l) = - \int\int M_l \, dx \, dx = - \frac{hx^2}{2} + c_2 x + c_4.$$

The constants of integration c_1 to c_4 are calculated from the following conditions.

(a) At the middle of the span l the tangent to the deflection-curve is horizontal if the structure is symmetrical:

$$[\phi_l]_{x=b+\frac{l}{2}} = 0.$$

Hence $c_2 = \frac{h}{2}(2b + l)$.

(b) At B the elastic curve is continuous, that is

$$[\phi_l]_{x=b} = [\phi_b]_{x=b}.$$

Consequently $c_1 = \frac{h}{2}(b + l)$.

(c) There is no vertical displacement of the intersection of a straight line joining the points of support A and B and a vertical line passing through the hinge at the base. Also, the effect of axial deformation of the inclined supports is neglected. Given these conditions the horizontal movements of A and B have to be in an opposing sense and their values are proportional to their horizontal distances from the hinges at the base, thus

$$\frac{[y_b]_{x=0}}{(1 - \alpha)b} = - \frac{[y_b]_{x=b}}{\alpha b}.$$

Consequently

$$c_3 = - (1 - \alpha) \frac{hb}{6}(2b + 3l).$$

(d) At B the curve of deflection is continuous, hence

$$[y_l]_{x=b} = [y_b]_{x=b}.$$

Consequently,

$$c_4 = \frac{hb^2}{6} - (1 - \alpha)\frac{hb}{6}(2b + 3l).$$

With these values for the constants of integration the equations of deformation become

$$(y_b) = (a_{01})_b = - \frac{h}{6b}\{x^3 - 3b(b + l)x + (1 - \alpha)b^2(2b + 3l)\}.$$

$$(y_l) = (a_{01})_l = - \frac{h}{6}\{3x^2 - 3(2b + l)x + (1 - \alpha)b(2b + 3l) + b^2\}$$

Calculation of a_{11}.

According to the relations determined by the bending moment diagram in *Fig.* 2,

$$(a_{11}) = \frac{h^2}{3}(2b + 3l).$$

Consequently the equations of the influence lines for the two sections are

$$\eta_b = \frac{x^3 - 3b(b + l)x + (1 - \alpha)b^2(2b + 3l)}{2bh(2b + 3l)};$$

$$\eta_l = \frac{3x^2 - 3(2b + l)x + (1 - \alpha)b(2b + 3l) + b^2}{2h(2b + 3l)}.$$

These equations determine the influence line for the horizontal force H from which may be determined the influence lines for the internal forces, bending moments, and shearing forces, and hence the stresses at any point in the structure may be calculated. By this method of calculation it is simple to determine the stresses in any statically-indeterminate structure supported on inclined columns. As previously mentioned, the shortening or lengthening of the inclined supports is neglected as the effect on the forces in the frame is generally negligible; nevertheless, these deformations may easily be accounted for in the calculations if this is considered necessary.

Statical Analysis.

In the design of a structure with inclined columns there is a much greater freedom of choice of dimensions than there is with the usual method of construction in which, for example, the choice is limited to determining the spans from an overall length of a bridge. Because of this, the analysis of frames with inclined columns

is limited to the single case of a two-pinned frame with cantilevers in which the possible variations include the lengths of the cantilevers, the distance between the inclined members and hence the nominal span, and the real span of the frame which is the distance between the hinges at the base. Three cases are shown in *Fig.* 3, in which the nominal span is assumed to be unity and the over-all length of the frame 2·5 units. The equations for the influence lines for H have been calculated for these cases.

Fig. 3*a*: The outer supports are vertical and $\alpha = 1$:

$$\eta_b = 0{\cdot}5x^3 - 1{\cdot}125x.$$

$$\eta_l = 0{\cdot}75x^2 - 1{\cdot}5x + 0{\cdot}065.$$

Fig. 3*b*: $\alpha = 0{\cdot}5$ where the supports form equilateral triangles:

$$\eta_b = 0{\cdot}5x^3 - 1{\cdot}125x + 0{\cdot}25.$$

$$\eta_l = 0{\cdot}75x^2 - 1{\cdot}5x + 0{\cdot}3125.$$

Fig. 3*c*: $\alpha = 0$ and the interior supports are vertical:

$$\eta_b = 0{\cdot}5x^3 - 1{\cdot}125x + 0{\cdot}5.$$

$$\eta_l = 0{\cdot}75x^2 - 1{\cdot}5x + 0{\cdot}5625.$$

The influence lines for the three cases are the same except that the base-line is varied as shown in *Fig.* 4. It may therefore be seen that by altering the real span of the frame the positive and negative parts of the bending-moment diagram may be altered to give the most favourable values. For example, in the case of a frame in which the dead load (assumed to be uniformly distributed) is preponderant the positions of the hinges at the bases would be chosen so that the total area of the bending-moment diagram approaches zero, and hence for the dead load the forces would be vertical only, that is H would be zero. It is frequently of importance with certain foundations that this should be so. The only horizontal force on the base would be that due to the imposed loads and would be of secondary importance in the design of the foundations.

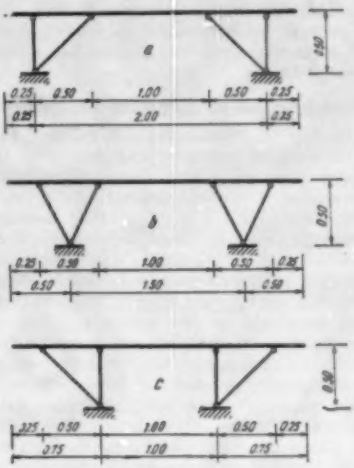


Fig. 3.

Fig. 4 shows also the influence line for H for a three-pinned frame (shown in a

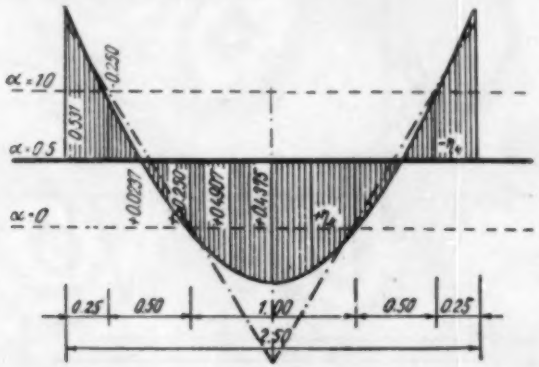


Fig. 4.

chain-dotted line) and it may be seen that in general the horizontal thrust is greater for a three-pinned frame than for a two-pinned frame.

Comparison of Bending Moments.

In *Fig.* 5 are shown two frames, one with vertical columns and one with inclined columns. For these frames the envelopes of maximum bending moments are shown in *Fig.* 6; the bending moments due to dead loads only are shown by a dotted line. The maximum bending moments are those due to dead and live loads, variation in temperature, and shrinkage of the concrete. Due to the rigidity of the inclined columns the effects of changes of temperature and of shrinkage are greater for the frame with these columns than for the frame with vertical columns. Comparing the bending moments in the two forms of construction, the maximum positive moment in the normal frame is 365 tonnes-metres against 265 tonnes-metres for a frame with inclined columns; similarly the comparative negative bending moments are 579 tonnes-metres and 288 tonnes-metres. That is, the positive bending moments have been reduced by 27 per cent. and the negative bending moments by 50 per cent., while the total area of the bending moment envelope has been reduced by 40 per cent. Such reductions have a considerable influence on the sizes of the members and consequently on the dead load, but the bending moments given do not take this into account as both cases have been calculated for the same loads.

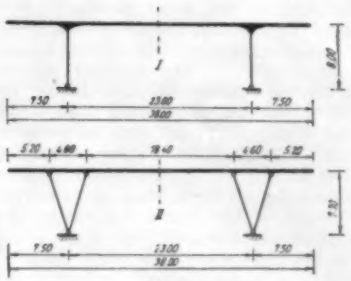


Fig. 5.

The shearing forces are also reduced, particularly in the part of the beam between the inclined supports where the shearing force is greatest in the general case.

Displacement of the Supports, and Deflections.

Neither the frame with vertical columns nor the frame with V-shaped supports is sensitive to vertical movements, but horizontal displacement has a greater effect on the frame with V-shaped supports than on the normal frame.

Deflections of the V-frame are less than those of the normal frame, as may be seen from *Fig.* 5. In bridges this is important, particularly in relation to the joint between the ends of the cantilevers and the road slab. Frequently the length of a cantilever is less than about 24 ft. as otherwise the deflections under a moving load may damage the surface. In the

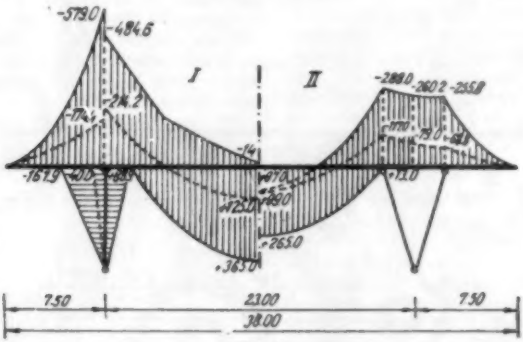


Fig. 6.

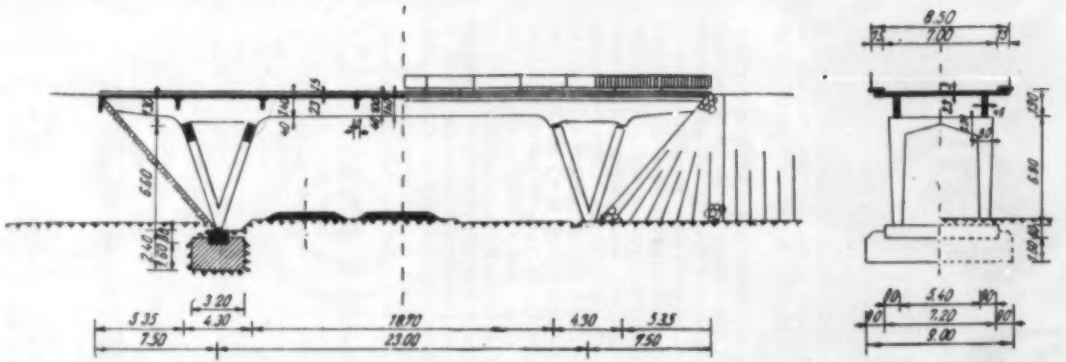


Fig. 7.—Road Bridge with Supports Hinged at Main Beams.

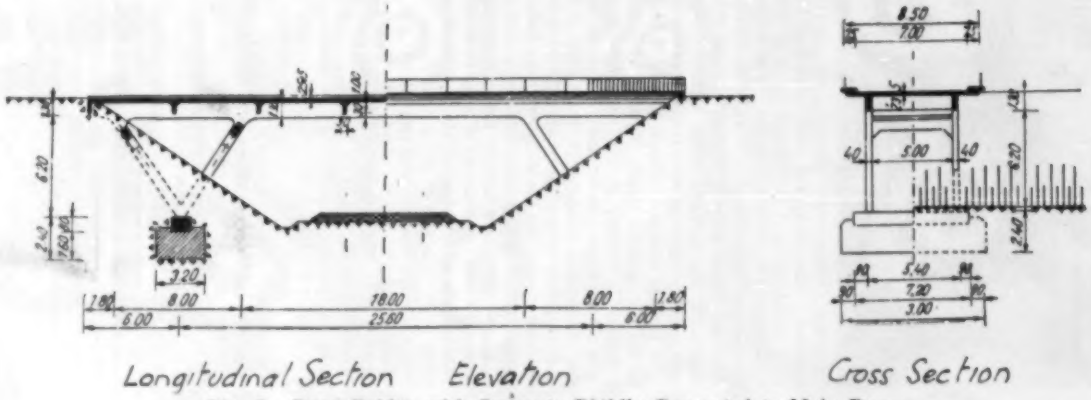


Fig. 8.—Road Bridge with Supports Rigidly Connected to Main Beams.

construction proposed in *Fig.* 5, for example, it is possible to reduce the length of the cantilever from 24 ft. to 16 ft., and as the deflection is proportional to the cube of the span the deflection in this case is reduced by one-third.

Examples of Bridges with V-shaped Supports.

Fig. 7 shows a bridge carrying a road over a railway cutting. As in the case of the frame for which the bending moments have been calculated, this bridge consists of two main beams at about 18-ft. centres carrying transverse beams at 16-ft. centres; the deck is a slab spanning in two directions and cantilevering over the main beams to form footpaths. The main beams are supported on inclined columns forming three-pinned frames across the width of the bridge. This arrangement was adopted to resist lateral wind pressure.

A similar bridge is shown in *Fig.* 8 but here the greater part of the inclined columns is buried in the embankment. The two bridges are the same in principle, the difference being that in *Fig.* 7 the supports are hinged at their intersection with the main beams while in *Fig.* 8 the supports are continuous with the main beams. In the latter case bending moments are produced in the inclined columns but, because of their flexibility compared with the beams, the bending moments are small (they can easily be determined by the moment-distribution method).

Architectural Aspects.

For structures such as these it is not only necessary to determine the structural advantages of using V-shaped supports but also to consider the appearance of the structures. Elevations are shown in *Figs.* 9 and 10 of the two bridges of which the structural details are given in *Figs.* 7 and 8. It is not assumed that these sketches show the best æsthetic development of the proposed method, but they compare favourably with other methods.

Fig. 11 shows a road-bridge in Hungary. The bridge carries a carriageway 22 ft. wide and two footpaths each about 2 ft. 6 in. wide over a railway cutting. The angle of intersection of the road and the railway is about 60 deg. The bridge comprises two main beams, each about 4 ft. 3 in. deep at the centre, supported on inclined columns hinged at both top and bottom. The total length of the bridge is 132 ft., the cantilevers are about 28 ft. 9 in., and the actual span of the frame is 74 ft. 6 in. The length of the cantilevers was chosen so as to reduce as much as possible the horizontal forces at the bases due to the dead loads.

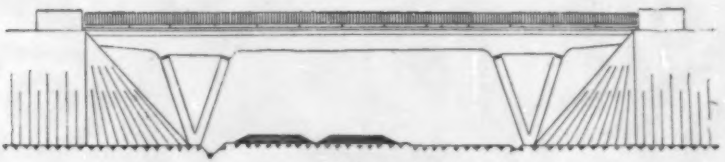


Fig. 9.

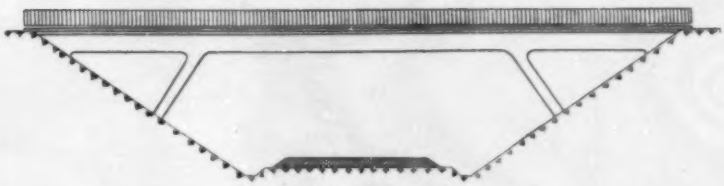


Fig. 10.

Fig. 11.—Bridge over a Railway.

The quantities of materials used were—concrete: 213 cu. yd. (0·52 cu. yd. per square yard); shuttering: 990 sq. yd. (2·45 sq. yd. per square yard); steel: 33 tons (182 lb. per square yard). Quantities of materials calculated for this bridge with vertical columns were about 10 per cent. greater.

Professional Status.

We have received the following letter from Mr. J. G. Orr, M.A., Secretary of the Engineers' Guild.

My attention has been drawn to the Editorial Note in your May issue, commenting on some remarks which I made at a meeting in Belfast during the Spring. Your comments can, I think, be summarised as follows: there is no doubt that engineering is accepted as a profession; if 30 per cent. of the directors of our engineering companies are engineers, this is a reasonable proportion and affords no legitimate ground for complaint; claims for higher status are likely to be no more than propaganda aimed at securing higher salaries, and this end is more likely to be attained by the formation of a new trade union.

If a sample poll were taken of the "man in the street" it is doubtful whether more than a very few would include engineering in a list of the leading professions. You ask, "Who has denied that engineering is a profession or that engineers are professional people?", but it is not necessary to do this for their status to suffer. The tendency to treat the professional engineer at all levels as a specialist assistant to the administrator is dying hard, and is bound to have its effect on the future expansion of the profession on which the country depends. This is not the view of the Guild alone, and the danger has been emphasised from many distinguished quarters. Professor J. F. Baker of Cambridge University wrote recently "Another most important reason [which may deter young men from entering the engineering profession] is the comparatively poor status of the engineer. For more than a century this country, unlike others, has not accorded its engineers the standing, financial or otherwise, which the responsibilities and difficulties of their work warrant."

The point which I made about the composition of the directorates of companies in the engineering industry is, I feel, an excellent illustration of the effect of this traditional attitude in practice. The term "technically qualified" included all those directors who were members of one or other of fourteen named engineering institutions as well as those possessing university degrees in science or engineering, and also members of the accountancy profession; only 16 per cent. were chartered engineers. Furthermore, as might be expected, the number of directors with technical qualifications was not evenly distributed, and as many as 40 per cent.

of the companies had no technically qualified director. The engineering industry is one in the direction of which the professional engineer might reasonably expect to play a major part, and to say this is in no way to deny the place of those trained in the humanities nor to subscribe to the attitude which you make haste to condemn.

Whether the Guild or yourself is more in touch with the feelings of the majority of members of the profession on the importance of professional status, may be left with your readers to decide. I would only emphasise that whilst status and salary are closely related they are not interchangeable terms. Salaries markedly affect status but they are not the sum of it, in engineering or any other profession. Finally you suggest " the formation of a strong trade union that can enforce [the professiona's] demands ". All the evidence points to the conclusion that the formation of yet more organisations does little to remedy existing troubles, and it is not clear in what way such a body would be better able than the Guild to protect the interests of professional engineers. Indeed, whether or not it is true that " doctors, headmasters and many other professional people . . . are already members of trade unions and do not seem to be unduly concerned because they adopt the tactics of manual workers when they collectively make demands for more pay for professional services ", it does not follow that the adoption of the same tactics would resolve the particular problems which confront the engineering profession at the present time.

[This letter is referred to on page 449 of this number, where the difficulty is pointed out of intelligently discussing a relative term such as " status " unless its meaning is defined. The foregoing letter does nothing to elucidate this problem, except to add to the claim for more directorships a complaint that engineering is not held in sufficiently high esteem by " the man in the street ". It also introduces a reference to " the particular problems which confront the engineering profession at the present time ", without defining the nature of those problems apart from the fewness of directorships and the alleged poor status.

We have every sympathy with engineers, and any other useful members of society, who wish to improve their position in life. The only difference between our correspondent and ourselves is that we do not agree that the kind of rhetoric used in the paragraph sent for publication in our May number, and seen again in parts of the foregoing letter, is the best means of achieving this end in the case of members of learned societies engaged in their profession.

In his first announcement our correspondent stated that in 725 of the biggest companies in the engineering industry about 70 per cent. of the directors had no technical qualifications. In its context this could only mean that about 30 per cent. had engineering qualifications. We now see that this statement was quite misleading because accountants were included amongst the technically qualified directors of engineering firms, and that only 16 per cent. are chartered engineers. Had we known that our correspondent's first figure was about 100 per cent. wrong our comment would have been different.

On the question of whether or not any of the members of the Guild are in favour of the formation of a trade union to promote their interests, we would refer our correspondent to the Journal of his Guild, which has published many letters, often vehemently expressed, advocating that the Guild become a registered trade union. Many of the members of the Guild employed in the Civil service, by local authorities, and by the nationalised industries are already members of trade unions. As these, together with members who are employers, would not be expected to desire another trade union, it is likely that the members who urged the Guild to become a trade union were working in circumstances in which they could not join one of the existing trade unions.—EDITOR.]

Moving a Four-story Building.

In the re-organisation of the factories at Oissel of the Compagnie Française des Materières Colorantes a comparison of the estimated cost of constructing a new building and the cost of moving an unused building from another part of the site showed a considerable saving in favour of the latter scheme. The building is a four-story structure about 326 ft. long by 35 ft. 6 in. wide, built of reinforced concrete flat-slab construction and weighing about 5000 tons. A joint across the width of the building divided it into halves; one half was successfully moved during 1955, and the second half will be moved after the installation of machinery in the first half.

Fig. 1 shows the complete building. The total length of movement was about a kilometre and included a 3 per cent. fall over a length of 680 ft., a rise of 2 per cent. over 250 ft., and rotational movements of 44 deg. and 65 deg.

Each half of the building is supported

Fig. 1.—Building Before Removal.

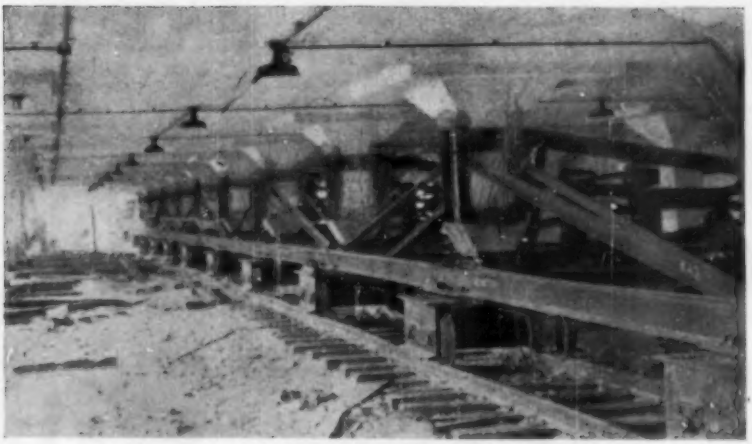


Fig. 2.—Underside of Building During Removal.

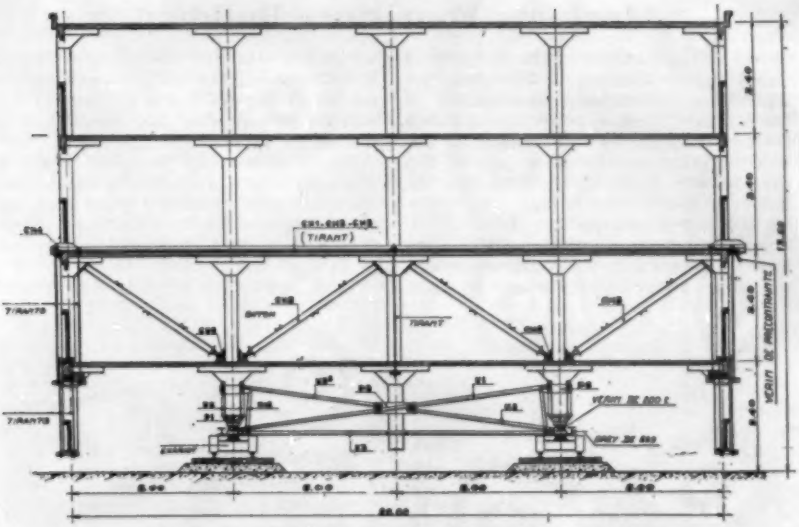


Fig. 3.—Cross Section showing Bracing and Trollies.

by 55 columns equally spaced in five rows. In order to support the building on two lines of rails transverse stiffening frames were built between the first and second stories across each line of columns, as shown in *Fig.* 3. These frames were supported under the first internal rows of columns, cantilevering beyond them to carry the external columns and carrying the central row of columns in the middle of their spans. The first-floor slab formed the lower boom of the frames, the second-floor slab the upper boom, and the diagonal members were steel struts. Because of the cantilevers the upper boom was in tension and prestressed ties were used across the second floor to avoid high tensile stresses in the slab. The diagonal struts were also prestressed by hydraulic jacks at their lower ends. A vertical tie was used between the upper and lower booms at the central columns.

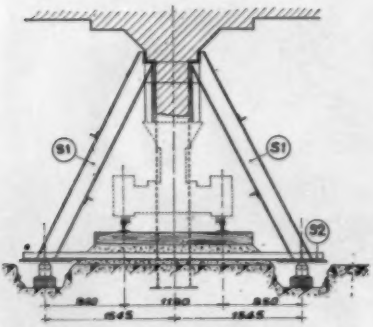


Fig. 4.—Method of Lifting Structure on to Trollies.

Each of the eleven frames was supported on two jacks of 200 tons capacity carried on four-wheeled trollies travelling on rails. The jacks were connected in groups so that it was possible to alter the relative level of groups and so control the forces in the structure during movement. The method adopted to mount the structure on the trollies is shown in *Fig.* 4. At each of the twenty-two columns an A-frame was formed of steel struts and a tie-bar; these struts were supported on small hydraulic jacks on each side of the rail-tracks so that the loads could be transferred from the bases to the A-frames. When this was done the columns could be cut away from the bases, the rail tracks laid, and the trollies positioned under the columns. The loads were then transferred to the trollies and the building was moved when the ties of the A-frames

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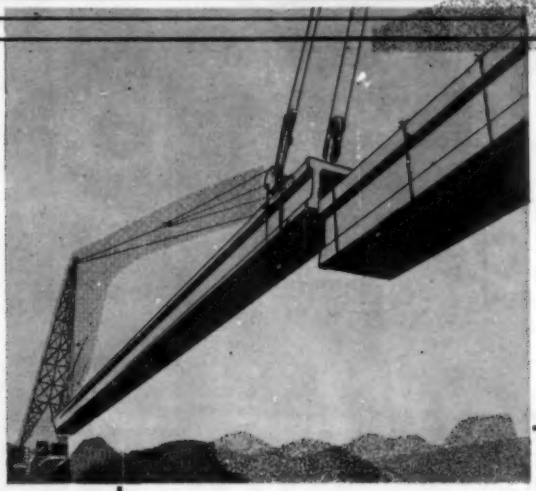

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were cut and the struts removed. The completed arrangement is shown in *Fig.* 2, in which can be seen the cross bracing to stiffen the structure.

Hydraulic jacks clamped to the rails and thrusting against the trollies were used to push the building along the tracks. Six jacks were used giving a total horizontal thrust of 300 tons; the thrust required varied from 75 tons to 300 tons depending on the inclination of the track. The pumps for the jacks were carried on trollies at the rear of the building. Telephones were used for communication between the men operating the pumps and those guiding the building and checking the movement.

The most difficult operation was to turn the building; this was done by lifting the structure by auxiliary jacks, on concrete bases, on to the eleven lines of curved track, then using jacks clamped to the rails to obtain the rotary motion. At the end of the curve the procedure was reversed.

The total time taken to move the structure was six months, of which four were for the actual movement. Along a straight line a speed of between 3 in. and 6 in. per minute was possible, the greatest distance traversed in a day being about 185 ft. It is estimated that, with the experience gained with the first half, moving the second half will take only 2½ months. The work is fully described by M. Kurt Olsen and M. Jacques Prévost in the French journal "Travaux" for February, 1956. Messrs. Christiani et Nielsen, who developed the methods, were the contractors.

Limiting the Effects of Dewatering Soil.

THE site of five new blocks of flats, fourteen stories high, at Coney Island, New York, extends along the sea front. At each end of the site (*Fig.* 1) are buildings, one of which is fifteen stories high. Because the subsoil would not support the new buildings on either normal foundations or piles, it was decided to make use of the buoyancy of a foundation of concrete boxes founded 10 ft. below sea-level. To construct the foundations it was necessary to lower the level of the ground-water by dewatering the site, but it was thought that this might cause settlement of the existing buildings. Tests showed that the effect of dewatering would extend 1000 ft. from the well-points. It was therefore decided to maintain the water level outside the boundary of the site by pumping water back into the ground beyond the boundary of the new building site.

The equipment used for replacing the water removed from the site of the

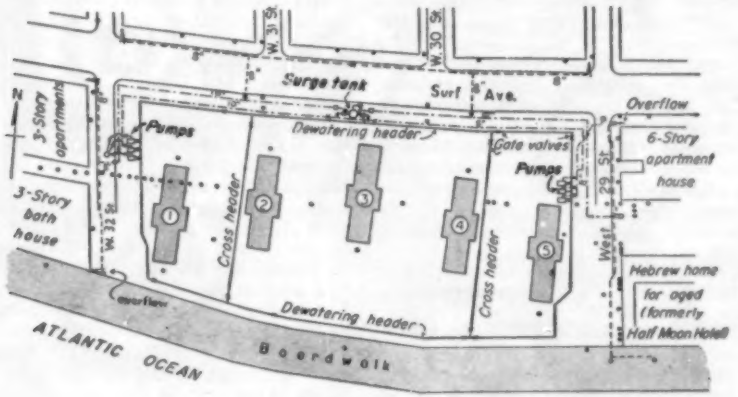

Fig. 1.

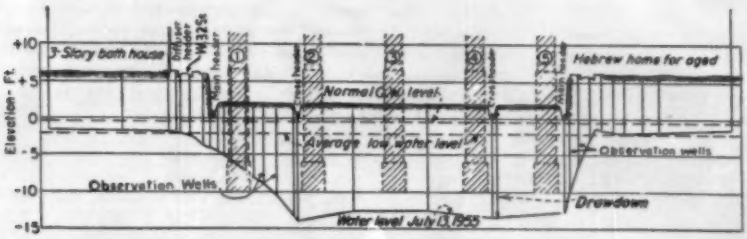

Fig. 2.

existing buildings was similar to that used for dewatering. The diffusion points had screens 8 ft. long compared with screens 3 ft. long for the well-points to provide a larger area for the discharge. The diffusion points were jetted into the ground in a similar manner to the well-points. In a preliminary test the diffusion points became clogged with a precipitate of ferrous hydroxide, which could not easily be filtered. This difficulty was reduced by placing zinc rods at from 4 ft. to 6 ft. centres in the pipelines in order to provide an electrical potential to retard the galvanic action between the metals in the water. The pipes were coated with vinyl paint, the diffusion risers were galvanised, and the screens were insulated to prevent electrolytic action.

The diffusion points were fed by gravity from a surge-tank 25 ft. high the water in which was maintained at the required level by water discharged from the well-points. The maximum pressure in the diffusion pipeline was controlled at 3 lb. per square inch by vented overflow loops 7 ft. high. By these means the ground water level below the site was reduced with little effect on the surrounding area, as shown in *Fig.* 2.

The consulting engineers were Messrs. Moran, Proctor, Mueser, and Routledge. The dewatering and diffusion systems were designed by the Griffin Wellpoint Corporation and were installed by the Wellpoint Dewatering Corporation. [The foregoing is abstracted from " Engineering News-Record " for April 12, 1956.]

New Works on British Railways.

AMONG the new works that the British Transport Commission proposes to start in the years 1956 and 1957 are the following.

Three tunnels between New Barnet and Potters Bar, estimated to cost £2,750,000.

Two branch lines to serve collieries at Bevercotes and Cotgrave, Notts, at a cost of about £1,500,000. New lines to serve collieries at Glenochil in Clackmannan and at Killock in Ayrshire, at a total cost of about £300,000.

A bridge south of Bletchley station at a cost of £1,250,000.

A rail-welding and track-assembly plant at Dinsdale and a new signal engineering works at York.

The electrification of the line from Fenchurch Street (London) to Tilbury and Southend, including the reconstruction of Barking station and the construction of bridges.

The electrification of the suburban lines from Liverpool Street (London) to Enfield, Chingford, Hertford, and Bishop's Stortford, at an estimated cost of £6,500,000. The electrification of the line between Manchester and Crewe.

Reconstruction or improvement of the following stations : Barrow Central, Cannon Street (London), Banbury, Glasgow Central, Chichester, Weymouth, Forest Hill, and Plymouth (North Road).

The provision of escalators and deep-level subways at the Bank (London) station (Waterloo and City Railways), at an estimated cost of nearly £1,050,000.

A station at Harlow.

A wagon repair depot between Thornton and Alloa at a cost of nearly £400,000.

Improvement of depots at Simonside (Co. Durham), Newport (Mon.), and Neath, at a cost of £700,000.

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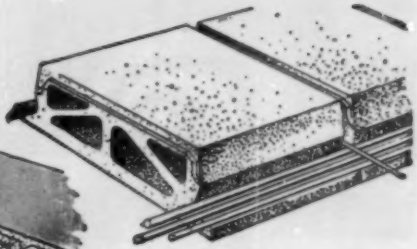

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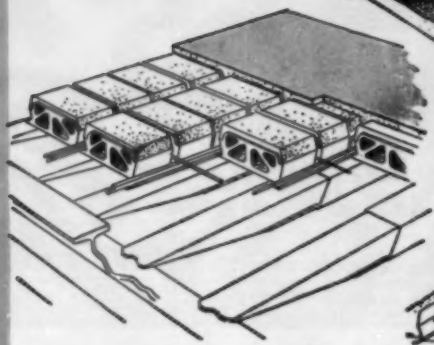

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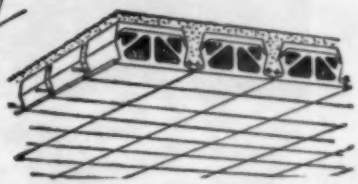

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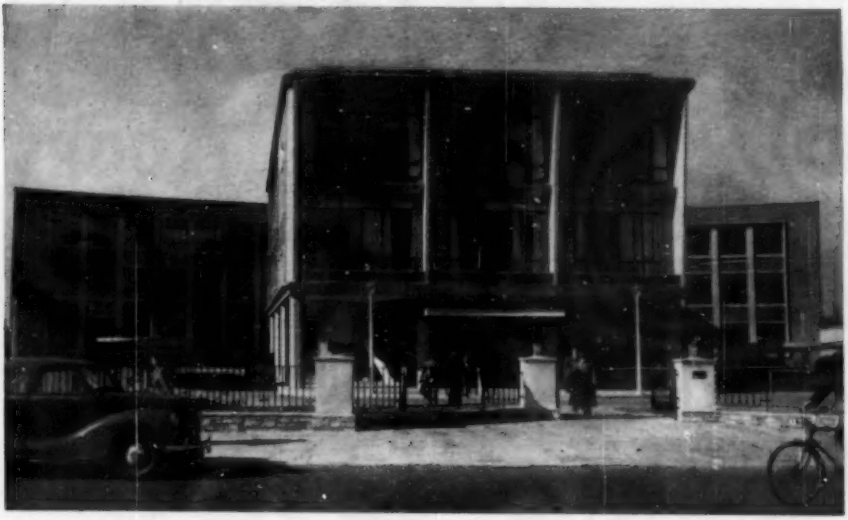

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A Building Tilted by the Wind.

A GARAGE (*Fig.* 1) of eight stories, comprising two parts each 91 ft. by 21 ft. in plan, under construction at Cleveland, U.S.A., was recently blown 6½ deg. out of plumb by winds with gusts of 35 to 50 miles per hour. The garage was being constructed by the "lift-slab" method and at the end of the previous day all the slabs, measuring 91 ft. by 21 ft. by 8 in. thick, had been raised to the required levels. The slabs were supported by ten hollow steel columns 61 ft. high made from two 8-in. by 8-in. angles welded together. There are ten columns spaced at 22 ft. centres in each of two rows, and up to a height of 11 ft. above the ground floor they were strengthened by two 5-in. by $\frac{5}{16}$-in. plates attached to the faces of the columns in the longer direction of the building. Cast-steel collars surrounding the columns were built into the slabs and the slabs were held temporarily in position by steel wedges below the collars. It is stated that the usual practice is to secure the slabs permanently by welding steel plates to the columns below the collar immediately the slabs have been positioned, but in this case it was proposed to weld the connections after all the slabs had been positioned. It was stated by the Cleveland Building Commissioner that had the usual practice been followed the building would have been sufficiently rigid to withstand the wind. It is reported in "Engineering News-Record" from which the foregoing notes and the illustration are taken that the only obvious damage was cracks 2 ft. long in three slabs near the collars by which they were fixed to the posts.

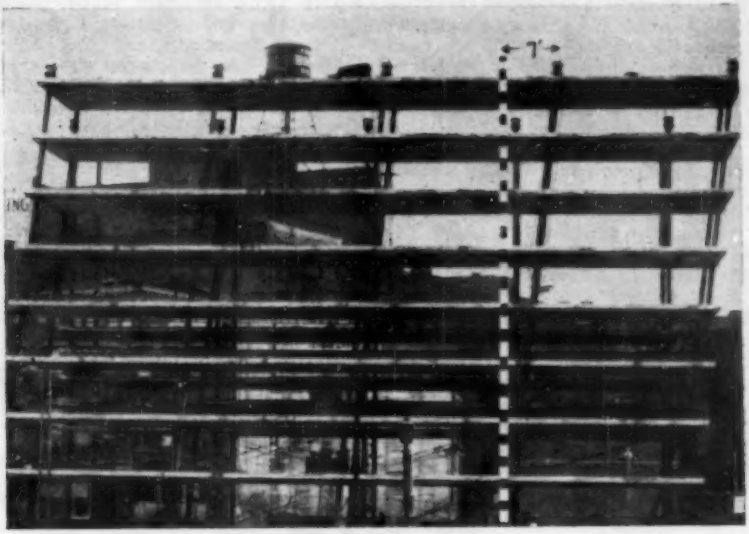

Fig. 1.

The Shrinkage of Concrete.

WHEN a concrete building 275 ft. long was built without joints at Rochester, New York, U.S.A., in the year 1939, electric strain meters were inserted in the four floor slabs and in the columns. After sixteen years the shrinkage has been about 0·6 in. in the floor slabs. Details of this test were given in a paper read by Mr. Robert R. Sheridan and Mr. Morgan B. Klock at the annual convention of the American Concrete Institute held at Philadelphia in February last.

The Direction of Research.

THE problems of how to choose subjects for research, when to stop a particular investigation, and the use of research staff, are to be discussed at an international conference arranged by the National Physical Laboratory to be held on September 26 to 28. The attendance will be restricted to directors and administrators of research. Details of the conference may be had from the Secretary, National Physical Laboratory, Teddington, Middlesex. The programme includes the following papers.

" Fundamental and Applied Aspects of Research Problems ", by Professor J. D. Bernal (Birkbeck College, University of London).

" The Research Project—Choice and Termination ", by Dr. Willis Jackson (Metropolitan-Vickers Electrical Co., Ltd.).

" Personality and Social Factors Related to Creativity ", by Professor Morris Stein (University of Chicago).

" Budgets and Administrative Controls ", by Mr. D. R. Willson, M.B.E. (Atomic Research Establishment, Harwell).

" Some Factors Affecting Laboratory Administration ", by Mr. E. S. Hiscocks (National Physical Laboratory).

" Science and Workshops ", by Professor Dr. R. Vieweg and Professor Dr. Scheibe (Physikalisch-Technische Bundesanstalt).

" Building for Research—What is Required ", by Dr. F. M. Lea, C.B.E. (Department of Scientific and Industrial Research).

" Building for Research—Design, Construction, and Layout of Laboratories ", by Mr. H. A. Snow (Ministry of Works).

" Staff Selection and Assessment ", by Dr. W. S. Bristowe (Imperial Chemical Industries, Ltd.) and by Dr. F. T. Rosser (National Research Council of Canada).

" Superiors and Subordinates in Research ", by Professor H. A. Shepard (Massachusetts Institute of Technology).

" Staff Groupings and the Flow of Authority ", by Sir Charles R. Harington (National Institute for Medical Research).

" Incentives ", by Mr. Stanley Mayne (Institution of Professional Civil Servants).

" Internal Communication ", by Dr. C. G. Williams (" Shell " Research, Ltd.).

" Studies in the Use of Technical Information in Smaller Industrial Establishments ", by Mr. Louis Moss and Mr. L. T. Wilkins (Central Office of Information).

" Knowledge : Passing it on—Getting it used ", by Dr. A. T. Green, O.B.E., and Dr. A. E. Dodd (British Ceramic Research Association).

" Some Fundamental Difficulties in Popularising Science ", by Mr. M. W. Thistle (National Research Council of Canada).

" Extramural Communications ", by Mr. A. J. Garratt, M.B.E. (National Physical Laboratory).

" The Assessment of the Work of a Laboratory ", by Mr. A. H. Wilson (Courtaulds, Ltd.).

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A NEW method of producing cellular concrete has recently been introduced into this country from Scandinavia, where it has been in use for the past fifteen years. It is primarily intended for producing a topping on floors and roofs requiring insulation against heat and sound, and for the insulation of pipes and the manufacture of precast blocks and slabs.

The concrete is made by mixing sand and cement with an air-entraining emulsion in a special type of mixer. It is claimed that concrete with densities of from 19 lb. to 75 lb. per cubic foot can be obtained with crushing strengths varying from 70 lb. to 750 lb. per square inch and a coefficient of thermal conductivity varying from 0·48 to 2·10. The air cells in the concrete are independent of each other and accordingly the concrete absorbs little water. For this reason it is claimed to have a high resistance to frost. The concrete is placed without vibration, which would tend to collapse the small air bubbles. It is very workable, however, and is screeded by drawing a board over the surface without tamping. This type of concrete is known as " Isocrete " and is produced by the Isocrete Co., Ltd.

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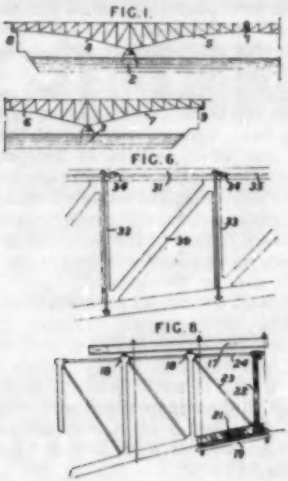
Patents Relating to Concrete.

Construction of Cantilevered Bridge Trusses.

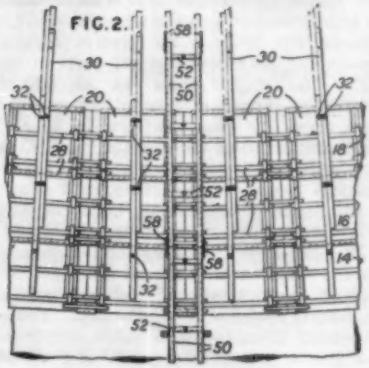
In a multiple-span reinforced concrete bridge (*Fig.* 1) the spans are supported by cantilever trusses (4, 5, 6, 7) which extend symmetrically from foundation pillars (2, 3) in both directions, the opposed ends of adjacent cantilevers being separated by a gap (1) and vertically interconnected by links, while the ends of the bridge are vertically supported on foundations (8, 9) by ordinary means (for example, roller bearings). In construction (*Fig.* 8) the reinforcement of the first two bays of each cantilever above each of the pillars is erected according to ordinary structural steel practice and the members in compression [the lower chord (21) and the vertical posts (22)] are embedded in concrete, while the members in tension [the upper chord (24) and the diagonals (23)] are left uncovered. An auxiliary movable scaffold comprising parallel beams (17) distributed over the width of the bridge and resting on the last two completed junctions (18) is erected to project beyond the end bay by a distance corresponding to one additional bay, and supports scaffolding and shuttering (19) whereby the reinforcement of the following bay is erected and sleeve-jointed to the completed construction. The members in compression are embedded in concrete and the scaffolding shifted for the construction of the following bay. When a cantilever is completed, the members in tension are prestressed by the vertical load and are then embedded in concrete.

In a modification (*Fig.* 6) the posts (33) on the upper chord (31) are in tension while the diagonals (30) and the lower chord are in compression. The short reinforcement bars of the posts and the upper chord are anchored at each junction, while the upper chord also contains other long bars (35) which extend the whole length of the bridge, to which their ends are anchored. The members in tension are prevented from bonding with the concrete by, for example, a coat of plastic material, and after each bay has been constructed and the concrete in which the reinforcement is embedded has set, a predetermined tension is applied to the short bars, and when the cantilever is complete the long bars are prestressed to prevent any tension being exerted on the concrete in the upper chord by the maximum load on the completed bridge. The reinforcement of the members in tension may be in bores in the concrete, which are later filled with grout.—No. 661,207. R. Finsterwalder and R. Jecht. April 21, 1949.



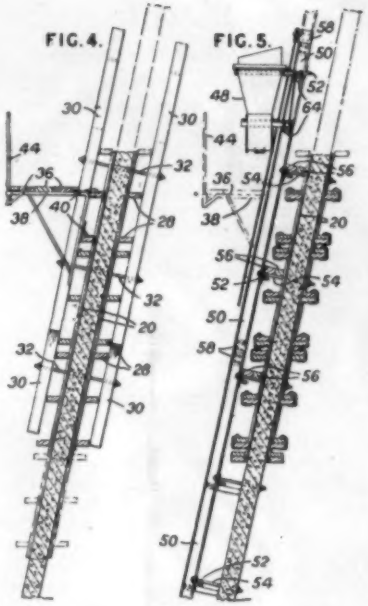
Hyperbolic Cooling Tower.

Walls are cast in situ between creeping shuttering on each side connected by cross-bolts, and scaffold brackets are removably fixed to the shutters; the hoist



for the materials also travels on guides secured to the shutters and to the finished wall. The arrangement is described with respect to shuttering for a cylindrical or conical tower, and comprises outer and inner shutterings (20) stiffened by horizontals and verticals (28, 30) and connected by cross-bolts (32), the scaffold brackets (38) having hooked portions (40) to engage over the horizontals (28). The brackets support boards (36) and a back-rail (44). The shutters are shown in three belts (14, 16, 18) so that, after one casting operation, the lower belt (14) may be removed and superposed on the upper belt (18). Channel guide-rails (50) for the hoist (48) are also fixed to the shuttering by bolts (54, *Fig.* 5), distance-pieces (56) being used to separate the shutters and the rails. Wheels (64) on the hoist (48) engage the guide-rails so that material can be raised to the casting position. Angle connectors (52) are disposed between the rails, and lengths are superposed and connected by fish-plates (58). —No. 520,400. Tileman & Co., Ltd., and E. Hansen. October 1948.

[Publication of patent specifications by the Patent Office has been much delayed due to the war.]



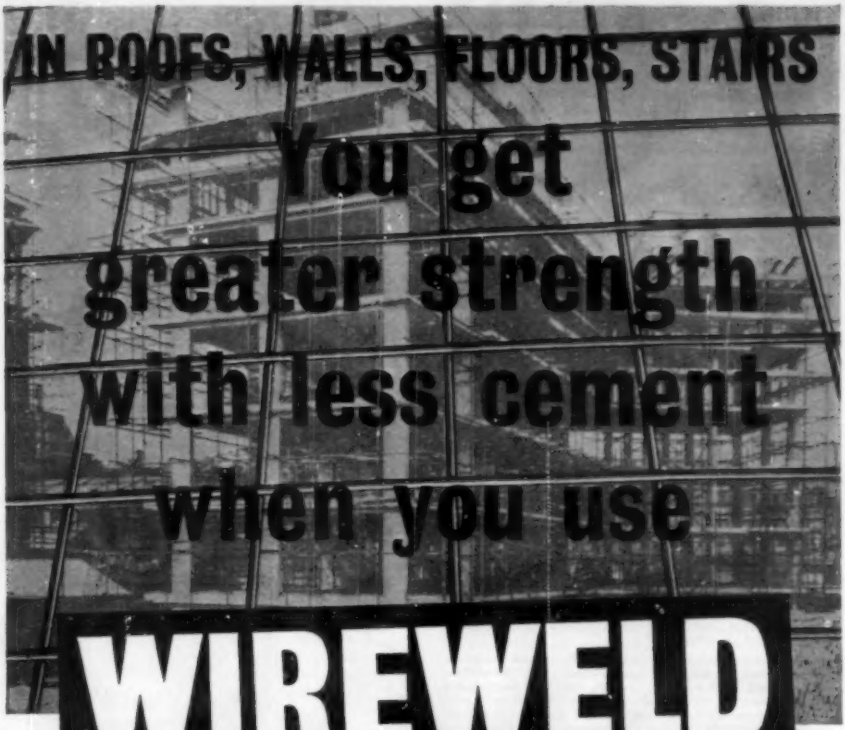
Asphalt Surfaces on Concrete Bases.

THE results of investigations made by the Asphalt Roads Association, in conjunction with the Cement and Concrete Association, show that a continuously reinforced concrete slab without expansion or contraction joints is most suitable as a concrete base for hot rolled asphalt, for the following reasons: (1) The presence of reinforcement and the absence of joints ensure that there is no relative vertical movement between adjacent parts of the slabs that might cause the asphalt to crack; (2) If cracks occur in the reinforced base these are small and are unlikely to be reproduced in the surface; (3) Continuously reinforced concrete is an economical form of construction, as it allows a minimum thickness of concrete and delays due to forming joints are eliminated.

It is recommended that the design and construction of a concrete base for an asphalt surface should follow normal reinforced concrete road practice. Reinforcement should pass through construction joints, and on a poor soil it may be advisable to include tie-bars or extra reinforcement as an additional precaution against the opening of cracks and relative vertical movement. Tie-bars should always be provided across longitudinal joints.

The greater the thickness of the asphalt surface, the better will be the performance of the base, because warping of the concrete due to variations in temperature through the depth of the slab are reduced as the thickness of the surface is increased. If the recommendations given are adopted the risk of failure of the concrete in compression during hot weather is so small that it can be disregarded.

Although satisfactory results have been obtained with asphalt surfaces 1½ in. thick it is recommended that they should be not less than 2 in. thick. When heavy dense traffic is anticipated it is recommended that the asphalt should comprise a 2-in. base and a 1½-in. wearing surface laid in accordance with British Standard No. 594.



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SITUATION VACANT. Civil engineering draughtsman required for work in London office of large civil engineering contractors. Work of varied nature, with scope for initiative and advancement. Commencing salary £600–£800 depending on experience. Five-days' week and luncheon vouchers. Pension scheme operates. Box 4228, Concrete and Constructional Engineering, 14 Dartmouth Street, London, S.W.1.

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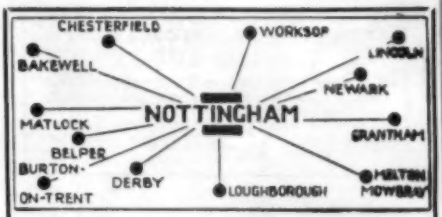

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